University of Southern Queensland

Faculty of Engineering and Surveying

Local and post-local buckling of steel tubes in thin-walled circular concrete-filled steel tubular columns

A dissertation submitted by

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Abstract

The local and post-local buckling behaviour of steel tubes in thin-walled circular concrete-filled steel tubular columns under axial load is presented. Geometric and material nonlinear finite element analysis is conducted on steel tubes under uniform edge compression to determine critical local and post-local buckling strengths.

The local and post-local buckling of steel tubes reduces the ultimate loads of thin-walled circular CFST columns under axial compression. Limited experimental research on CFST columns has been conducted. The effects of local and post-local buckling on the behaviour of thin-walled circular CFST columns have not been adequately researched.

Three-dimensional finite element models are developed using the finite element technique. Initial geometric imperfections, residual stresses, material yielding and strain hardening are considered in the analysis. Based on the results obtained from the nonlinear finite element analysis, a set of design formulas is proposed for determining the critical local buckling loads and ultimate strengths of steel tubes in thin-walled circular CFST columns. The proposed design formulas are verified by comparisons with existing experimental results.

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Mr Craig Walker

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Nomenclature

B	= width of steel plate
D	= diameter of steel tube
E	= elastic modulus of steel
f_{sy}	= yield strength for steel
f_u	= failure stress
f_y	= yield stress
$f_{y(o.2)}$	= yield proof stress
h	= height of steel tube
J'_2	= Second invariant of the deviatoric stress tensor
k	= local buckling coefficient
$K(k)$	= material parameter, function of the hardening parameter
L	= length
n	= knee factor (25)
ν	= Poisson's ratio of steel
t	= thickness of steel tube
w	= lateral deflection at the tube centre
δ	= geometrical imperfection
λ_e	= modified cross-section slenderness limit
ε	= uniaxial strain
σ	= uniaxial stress

σ_{0l} = elastic buckling stress

$\sigma_{0.7}$ = stress corresponding to $E_{0.7}$

$\sigma_{11}, \sigma_{22}, \sigma_{33}$ = principal stresses

σ_a = maximum edge stress

σ_c = critical elastic buckling stress of tubes with imperfections

σ_p = post-local buckling reserve of strength of a steel tube

σ_u = ultimate stress of steel tube

σ_{vm} = von Mises stress

Chapter 1 INTRODUCTION

1.1 Background

Concrete-filled steel tubular (CFST) columns are used as primary axial load carrying members in many structural applications including high-rise buildings, bridges, piles and offshore structures. The structural properties of CFST columns include high strength, high ductility and high energy absorption capacity. The load carrying capacity and behaviour in compression, bending and shear are all superior to reinforced concrete (Zhong & Goode 2001).

Currently, there is no comprehensive design standard that can be used for the design of thin-walled CFST columns. Extensive research has been conducted on steel-concrete composite columns in which structural steel is encased in concrete. However, minimal research has been directed at thin-walled circular CFST columns (O'Shea & Bridge 2000). Therefore, extensive research is required on the local and post-local buckling behaviour of thin-walled circular CFST columns and that is the topic of this dissertation.

Thin-walled CFST columns are those which are designed to take account of the beneficial effect of the concrete restraint against local buckling of the steel tube. The concrete core effectively prevents the steel tube from inward local buckling resulting in a higher buckling mode (Bridge et al. 1995). The local buckling restraint provided by the concrete core therefore leads to an increase in the local buckling strength of the steel tube. Furthermore, the local buckling restraint combined with the confinement of the concrete core by the steel tube leads to an increase in the ultimate strength of the CFST column.

The reduction of the steel tube thickness in thin-walled CFST columns has the potential to significantly reduce construction costs. However, thin-wall CFST columns are susceptible to the local instability problem of thin-walled steel plates under compression and in-plane bending. The local buckling of steel tubes with geometric imperfections and residual stresses results in a reduction in the strength and ductility of members (Liang et al. 2006).

1.2 Application of CFST Columns

The use of CFST columns in contemporary construction is increasing throughout the world (Uy 2000). Thin-walled CFST columns have been used in many Australian high-rise buildings including Casselden Place and the Commonwealth Centre in Melbourne, the Riverside and Myer Centres in Adelaide and Market Plaza in Sydney (Watson & O’Brien 1990).

CFST columns are constructed using circular hollow steel sections filled with either normal or high strength concrete. In a CFST column the concrete is fully encased by the steel tube. A typical cross section is illustrated in Figure 1. The concrete core has the advantages of high compressive strength and stiffness while the steel tube has the advantages of high strength and ductility. CFST columns combine steel and concrete, resulting in a member that has the beneficial qualities of both materials.

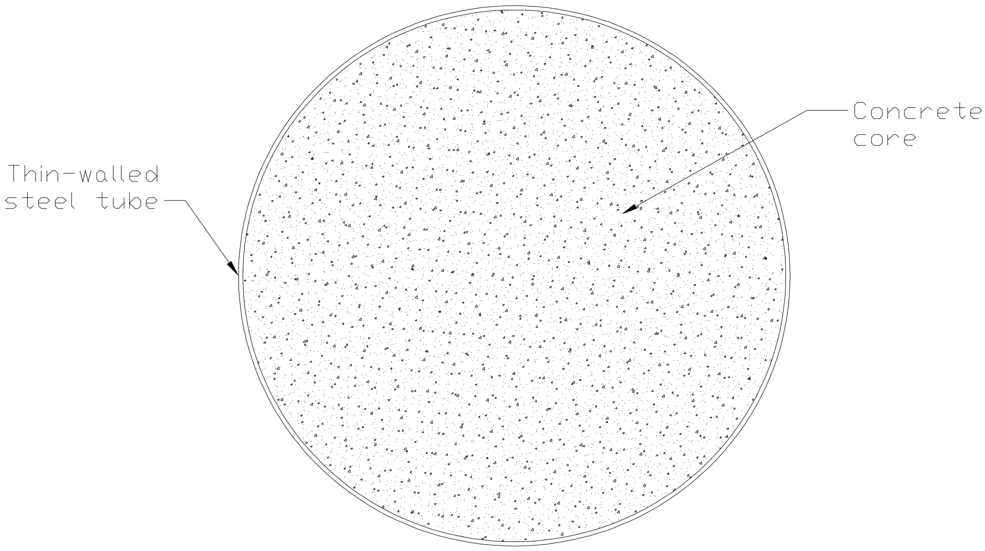

Figure 1.1: Cross section of CFST column.

Thin-walled CFST columns offer an economical solution for members subjected to primary axial loads due to the current increasing price of steel and the advantages offered during the construction process. The steel tube provides the necessary formwork and has the ability to support construction loads prior to placement of the concrete allowing for rapid construction. Furthermore, the increased compressive strength of CFST columns allows for smaller cross-sections resulting in lower material costs and an increase in lettable floor area and floor space flexibility.

1.3 Project Aims and Objectives

This project aims to investigate the local and post-local buckling behaviour of thin-walled circular CFST columns using the finite element method. The objectives of this project in accordance with the specification, as included in Appendix A, are as follows:

1. Research existing information relating to local and post-local buckling of concrete-filled thin-walled steel tubular columns.

2. Study the nonlinear finite element analysis method and develop three-dimensional finite element models for the nonlinear analysis of concrete-filled thin-walled steel tubular columns.

3. Conduct geometric and material nonlinear finite element analysis on steel tubes under uniform/non-uniform edge compression to determine critical load and post-local buckling strengths. Initial geometric imperfections, residual stresses, material yielding and strain hardening will be considered in the analysis.

4. Furthermore, investigate the effects of stress gradients and tube width-to-thickness ratios on the load-deflection curves and the post-local buckling strengths.

5. Based on the results obtained, propose a set of design formulas for determining the critical local buckling loads and ultimate strengths of concrete-filled thin-walled steel tubular columns.

6. Verify the proposed design formulas by comparison with existing experimental results and those developed by other researchers.

1.4 Structure of Dissertation

The investigation of the local and post-local buckling of steel tubes in thin-walled circular CFST columns involved reviews of related literature, nonlinear finite element modelling and analysis of results. This section outlines the structure of the dissertation.

Chapter 2 discusses the main findings of the literature review that was conducted on the local and post-local buckling behaviour of CFST columns. It includes both theoretical and experimental studies. A review and comparison of current design codes is also included.

Chapter 3 outlines the methodology used to analyse the local and post-local buckling of steel tubes in circular CFST columns. This includes the method used to propose the effective strength and ultimate strength formulas.

Chapter 4 provides a technical explanation of the finite element analysis phase, including the procedure undertaken to create the finite element model and conduct the analysis. An outline of the loading conditions, material properties, and model dimensions is also discussed.

Chapter 5 presents and discusses the results obtained throughout the analysis phase. The effects of geometric imperfections, slenderness rations, and height-to-diameter ratios on the local and post-local buckling behaviour of steel tubes in CFST columns were investigated. Effective strength and ultimate strength formulas were also proposed and verified.

Chapter 6 contains the final conclusion on the project analysis and results. This chapter also concludes on the aims and objectives of the project and provides recommendations for further study.

1.5 Summary

This dissertation aims to investigate the local and post-local buckling behaviour of thin-walled circular CFST columns using the finite element method. Effective strength and ultimate strength formulas for the design of steel tubes in CFST columns are also proposed. This investigation is necessary due to the lack of research directed at thin-walled circular CFST columns in the past and the increasing use of these members in contemporary construction.

Chapter 2 LITERATURE REVIEW

2.1 Introduction

This chapter will discuss published research that has investigated the behaviour of CFST columns. Extensive research has been conducted on steel-concrete composite columns in which structural steel is encased in concrete. However, CFST columns have received relatively little attention in comparison (Shanmugam & Lakshmi 2001).

Substantial effort has been aimed at developing a better understanding of the behaviour of CFST columns over the last 40 years (O'Shea and Bridge 2000). In this chapter, a review of the research conducted on CFST columns is presented with an emphasis on theoretical and experimental studies. A review and comparison of current design codes is also included. Furthermore, the review includes research work that has investigated the effects of various loading conditions, diameter-to-thickness ratios (D/t), length-to-diameter ratios (L/D), geometric imperfections and residual stresses on the local and post-local buckling characteristics, ultimate strengths, ductility and degree of confinement of CFST columns.

2.2 Theoretical Studies

2.2.1 Finite Element Analysis Studies

Hu et al. (2003) investigated the behaviour of CFST columns subjected to axial loads using the nonlinear finite element program *ABAQUS*. The cross-sections in the numerical analysis were categorised into three groups; circular section, square section, and square section stiffened by reinforcement ties. Based on the results for circular CFST columns, the steel tube can provide a large confining effect due to the concrete core, especially when the *D/t* ratio are small (*D/t* < 40), which indicated that local bucking of the steel tube is unlikely to occur. The results for square CFST columns suggested that the steel tube does not provide a large confining effect particularly when the width-to-thickness ratio is large (*B/t* >30). The results indicate that local buckling of the steel tube is very likely to occur. Based on the analysis results for square CFST columns stiffened by reinforcement ties, the confinement effect is enhanced especially when the tie spacing is small and the tie number or diameter is large. The results show that local buckling of the steel tube is prevented by the reinforcement tie. Furthermore, the results from the nonlinear finite element analysis indicate that the lateral confining pressure decreases with an increase in the *B/t* ratio due to the decrease in the lateral support for the steel tube.

Johansson and Gylitoft (41 2002) conducted an experimental and analytical study on the behaviour of CFST columns subjected to axial loading. The experiments included loading of the steel tube only, loading of the concrete only, simultaneous loading of the concrete and steel, and loading of hollow steel tubes. Test specimens had a *L/D* ratio of 4 and a D/t ratio of 33. Concrete strengths used in the test specimens achieved a mean compressive strength of 65 MPa. Nonlinear finite element models were created using *ABAQUS* and verified with the experimental results. The models were used to examine how the behaviour of the column was influenced by the bond strength between the steel tube and concrete core, and by the confinement effect offered by the steel tube. It was suggested by Johansson and Gylitoft (2002) that the following conclusions be drawn for the results. The load resistance of the short stub columns was determined by yielding of the steel tube. All of the columns sustained large displacements with almost completely maintained load resistance. The type of buckling mode at failure was greatly influenced by the method used to apply the load to the column section. Whether the load was applied to the entire section or only to the concrete section, the stability of the column was lost by a combination of local buckling and crushing. A pattern of inward and outward local buckles developed for hollow steel tubes, while ring buckles developed outwards only for the CFST column with the load applied to the steel. The bond strength had no influence on the behaviour when the steel and concrete sections were loaded simultaneously. However, it was noted that for columns with the load applied only to the concrete core, the bond strength highly affected the confinement effects and therefore the behaviour and ultimate strength of the CFST column. The degree of confinement effect is most pronounced for columns with no bond strength between the steel tube and concrete core when the load is applied to the concrete only. The stiffness was also influenced by the changed bond strength for this loading condition. Increased bond strength resulted in a greater contribution from the steel tube, meaning the stiffness increased. However, changing the bond strength had no effect for the columns with the load applied to the entire section or only to the steel section. Johansson and Gylitoft (2002) suggested that even though the efficiency of the steel tube in confining the concrete core is greater when the load is applied only to the concrete section, it seems unreliable to trust the natural bond strength to get full composite action. Therefore, it is recommended by Johansson and Gylitoft (2002) that the load be applied to the entire section.

Liang and Uy (2000) studied the post-local buckling behaviour of steel plates in thin-walled CFST welded box columns using the finite element method. The effects of various geometric imperfections, residual stresses and *B/t* ratios on the post-local buckling characteristic were investigated. A new method was developed for evaluating the initial local buckling loads and post-local buckling reserve strength of steel plates with imperfections based on the load-transverse deflection relations associated with the theoretical analysis. The accuracy of the design models were verified by a classical solution and experimental results. The results indicated that the theoretical predictions for the ultimate strength of steel plates and CFST box columns using the proposed design models agree very well with the experimental data. Therefore, Liang and Uy (2000) propose that the effective width formulas be used in the ultimate strength calculation of short thin-walled CFST box columns subjected to an axial load.

Liang et al. (2006a) performed geometric and material non-linear analysis, using the finite element analysis method, to investigate the critical local and post-local buckling strengths of steel plates in thin-walled CFST beam-columns under compression and in-plane bending. Clamped square plates with various *B/t* ratios, geometric imperfections and residual stresses were studied. Material yielding and strain hardening were also taken into account. The numerical results obtained by Liang et al. (2006a) indicated that increasing the *B/t* ratio of a steel plate under a predefined stress gradient reduces its lateral stiffness, critical local buckling stress and ultimate strength. The results also showed that the lateral stiffness, critical local buckling stress and ultimate strength under edge compression decreases with an increase in the stress gradient coefficient. Based on the results, a set of design formulas were developed for determining the critical local buckling and ultimate strength design of steel plates in CFST beam-columns. Additionally, effective width formulas were developed for the ultimate strength design of clamped steel plates under non-uniform compression. The proposed design formulas can be used directly in the design of CFST beam-columns and adopted in the advanced analysis to account for local buckling effects (Liang et al. 2006a).

Liang et al. (2006b) used the nonlinear fiber element analysis method to investigate the ultimate strength and behaviour of short thin-walled CFST box columns with local buckling effects. The confinement effect on the ductility of the encased concrete in CFST columns is included in the analysis. Effective width formulas proposed for steel plates in CFST box columns with geometric imperfections and residual stresses were incorporated in the fiber element analysis to account for local buckling. The effects of *B/t* ratios and compressive concrete strengths on the ultimate load and ductility of CFST box columns were investigated. Comparisons with existing experimental results indicated that the proposed fiber element analysis program predicted well the ultimate strengths and behaviour of CFST box columns with local buckling effects. Based on the results, increasing the *B/t* ratio reduces the ultimate strength, axial stiffness, section performance and ductility. Furthermore, increasing the compressive concrete strength increases the ultimate strength and axial stiffness of the CFST box column but reduces the section performance and ductility (Liang et al. 2006b). The procedure proposed by Liang et al. (2006b) for simulating the progressive local and post-local buckling of steel plates can be adopted in the nonlinear fiber element analysis of CFST columns under axial bending and biaxial bending. The fiber element program can also be used directly in design for predicting the ultimate strength and ductility of CFST box columns.

Liang et al. (2004) investigated the local and post-local buckling strength of steel plates in double skinned composite (DSC) panels under biaxial compression and in-plane shear by using the finite element method. Critical local buckling interaction relationships were shown for steel plates with various boundary conditions that included the shear stiffness effects of stud shear connectors. A geometric and material nonlinear analysis was used to study the post-local buckling interaction strength of steel plates. The strength interaction curves were generated using the proportional load incremental method in the nonlinear analysis for square plates with various *B/t* ratios. The finite element models developed incorporated initial imperfections of steel plates, material yielding, and nonlinear shear-slip behaviour of stud connectors. The results obtained by Liang et al. (2004) indicated that the ultimate strength of stocky plates is governed by the shear capacity of stud shear connectors or local buckling of steel plates. Furthermore, based on the results, slender steel plates can attain their full post-local buckling reserve of strength without the premature fracture of stud shear connectors. Design models for critical buckling and ultimate strength interactions were developed for determining the maximum stud spacing and ultimate strength in DSC panels. The maximum stud spacing predicted ensures that the critical local buckling of steel plates between stud shear connectors is prevented before the yielding of steel plates. The proposed design models can be used for the design of DSC panels with similar design situations.

2.2.2 Analytical Studies

Bradford et al. (2002) investigated the issue of local and post-local buckling of circular steel tubes with ridged infill, with the emphasis being on the strength of CFST sections used as composite columns. By using a Rayleigh-Ritz based method, a closed form solution for the local buckling stress of a thin-walled CFST column was derived, see Equation (1), and it was demonstrated that the elastic local buckling stress is $\sqrt{3}$ times that of its unfilled counterpart. The von Karman model of effective widths was interpreted in an effective width framework to determine the post-local buckling strength of a CFST column. The model showed that there is no post-local buckling reserve of strength, and that the critical buckling stress governs the strength of slender cross-sections. The use of this approach resulted in a prescriptive equation for the cross-section buckling strength of slender column; see Equations (2), (3) and (4).

$$\sigma_{0/} = \frac{2E}{\sqrt{1-v^2}} \frac{1}{(D/t)} \quad (1)$$

$$\lambda_e = (D/t)(f_y / f_{sy}) \quad (2)$$

$$\text{When } \lambda_e \leq 125 \qquad f_u = f_y \quad (3)$$

$$\text{When } \lambda_e > 125 \qquad f_u = 125 f_y / [(D/t)(f_y / f_{sy})] \quad (4)$$

The cross-section modified slenderness limit of 125, proposed by Bradford et al. (2002), is greater than the value of 100 proposed for hollow circular steel tubes. On the basis of this model, the cross-section is considered to be fully effective when $\lambda_e \leq 125$ and slender when $\lambda_e > 125$. It should be noted that the calibration technique adopted by Bradford et al. (2002) lacked reliable test data.

Lakshmi and Shanmugan (2002) used a semi-analytical method to investigate the inelastic and ultimate load behaviour of CFST columns subjected to uniaxial or biaxial loading in order to determine the ultimate strength of square, rectangular, and circular cross-sections. Moment-curvature-thrust relationships were generated for column cross-sections by an iterative process. Non-linear equilibrium equations resulting from geometric and material nonlinearities were solved by an incremental-iterative numerical scheme based on the Generalised Displacement Control method. The approach eliminated the limitation of the conventional analysis in which a deflected shape or pattern is assumed. However, the effects of local buckling of steel tube, residual stresses, strain hardening of steel, shear strain and tensile strength of concrete are neglected. The accuracy of the model was proven by comparison with existing experimental results. The results indicate that the moment capacity of columns decreases with an increase in applied axial load. Furthermore, columns loaded eccentrically displayed a significant decrease in moment capacity with an increase in eccentricity. Based on the results, the proposed analytical model could predict accurately the strength of CFST square, rectangular, and circular columns for a variety of column slenderness ratios. Therefore, Lakshmi and Shanmugan (2002) proposed that the method be used for direct analysis of short and slender composite columns subjected to uniaxial or biaxial bending.

Shanmugam and Lakshmi (2002) investigated the effect of local buckling on the axial compressive strength of thin-walled CFST box columns. A nonlinear analysis technique was used to formulate an incremental equilibrium equation based on the updated Lagrangian notation. The Generalised Displacement Control method was then applied to solve the equilibrium equation. The analytical method was extended in the study to thin-walled CFST columns. The model was calibrated using available experiment data and it was shown to be fairly accurate at predicting the behaviour and ultimate loads of thin-walled CFST box columns. However, the results from the analytical model appear to be conservative when compared with the experimental results. The results indicated that CFST box columns have greater strength than hollow steel box columns. Furthermore, the effect of local-bucking was found to be significant in thin-walled CFST columns.

2.3 Experimental Studies

2.3.1 Circular CFST Columns

O'Shea and Bridge (1997) conducted a comprehensive series of tests to examine the behaviour of short thin-wall circular steel tubes with or without internal restraint. The tubes had a *D/t* ratio ranging from 55-200 and a *L/D* ratio of 3.5. The experiments included bare steel loaded both axially and at small eccentricities, and axially loaded steel tubes with an internal restraint medium. The material properties were measured, including residual stresses and geometric imperfections. The test strengths were compared to strength models in design standards and design recommendations were proposed. The results indicated that local buckling significantly decreases the strength of circular thin-walled bare steel tubes and that concrete infill for circular steel tubes has little effect on the local buckling strength of steel tubes in axial compression. It was therefore suggested by O'Shea and Bridge (1997) that the strength of steel tubes in CFST columns be determined by using the design rules in current steel codes for bare steel tubes. However, it was noted by O'Shea and Bridge (1997) that concrete infill can improve the local buckling strength of rectangular and square steel tubes. The results also suggested that the provisions of current steel codes, which include the effects of local buckling on the section strength, are conservative for thin-walled bare steel tubes with small eccentric loads. This was especially evident when a linear interaction between the axial capacity and moment capacity was used.

O'Shea and Bridge (2000) performed a series of tests to study the behaviour of thin-wall circular CFST columns. The loading conditions included axial loading of the steel tube and simultaneous loading of the concrete and steel tube both axially and at small eccentricities. The test specimens had a *L/D* ratio of 3.5 and a *D/t* ratio ranging from 60 to 220. The internal concrete had nominal unconfined cylinder strengths of 50, 80, and 120 MPa. The results indicated that local buckling significantly affects the strength of unfilled steel tubes. Although the buckling strength of square steel tubes can be improved by providing internal lateral restraint, this was not observed in the circular test tubes examined by O'Shea and Bridge (2000). Instead, the predominantly outward buckling mode remained unaffected by the concrete core. The bond between the steel tube and the concrete core was found to be critical in determining the formation of a local buckle. O'Shea and Bridge (2000) state that local buckling of the steel tube will not occur for axially loaded thin-walled steel tubes if there is sufficient bond between the steel tube and concrete core. Furthermore, the degree of confinement offered by a thin-walled circular steel tube was dependent on the loading condition. The largest concrete confinement occurs with only the concrete core loaded axially and the steel tube used as pure circumferential restraint. Conservative design methods were developed that estimate the strength of thin-walled circular CFST columns subjected to different loading conditions. The design methods were calibrated and validated using previous test results on circular CFST columns and compared to Eurocode 4 (CEN 1992). The simplified methods used in Eurocode 4 (CEN 1992) can give very unconservative estimates if used outside their calibration range. Therefore, it was recommended by O'Shea and Bridge (2000) that Eurocode 4 (CEN 1992) only be used for the design of thin-walled steel tubes filled with very high strength concrete.

Giakoumelis and Lam (2004) investigated the behaviour of circular CFST columns subjected to axially compression with various concrete strengths. The effects of tube thickness, bond strength between the steel tube and concrete, and confinement of the concrete were studied. Test specimens had an *L/D* ratio of 3.5 and *D/t* ratios ranging from 22.9 to 20.5. The concrete infill had nominal unconfined cylinder strengths of 30, 60, and 100 MPa. The results show that the peak load was achieved with small displacement for high strength CFST columns and with large displacement for normal strength CFST columns. Based on the results, the effect of the bond between the steel tube and the concrete core becomes more critical as the concrete strength increases. For normal strength concrete, the reduction in the axial capacity of the column due to bonding was negligible. The results were compared with values estimated by Eurocode 4, Australian Standards, and American Codes. When the experimental results of Giakoumelis and Lam (2004) were compared to Eurocode 4, 17% was the largest variation between the experimental and calculated values for the axial capacity. The predicted axial strengths using AS4100 and ACI318 were 35% lower than the experimental results. Giakoumelis and Lam (2004) suggested that Eurocode 4 be used for the design of CFST columns with both normal and high strength concrete.

Fam et al. (2004) conducted experimental tests to investigate the behaviour of circular CFST beam-columns subjected to concentric axial compression loading and combined axial compression and lateral cyclic loading. The effects of different bond and end conditions on the strength and ductility of short CFST beam-columns were studied. Test specimens had a *L/D* ratio of 3 and a *D/t* ratio of 49. The concrete infill had an unconfined cylinder strength of 60 MPa. Both bonded and unbonded specimens were tested, including the application of the axial load to the CFST section and to the concrete core only. The results indicate that the bond and end loading conditions of a CFST beam-column do not significantly affect its flexural strength. However, the axially strengths of the unbonded columns were slightly greater than those of the bonded specimens due to the confinement effect. While the stiffness of the unbonded columns was slightly lower due to the absence of contribution of the steel tube in the axial direction. The behaviour of CFST columns under combined constant axial compression and lateral cyclic loads was very ductile. Bonded CFST columns exhibited better ductile behaviour than unbonded sections. Fam et al. (2004)

stated that current design standards significantly underestimate the maximum axial capacity of short CFST columns, including the flexural strength of CFST beam-columns subjected to axial compression and bending. Furthermore, an analytical model capable of predicting the flexural and axial load strength of CFST columns was presented.

2.3.2 Square CFST Columns

Uy (1998) conducted an extensive set of experiments on the local and post-local buckling of CFST box columns. The *B/t* ratios ranged from 40 to 100 and the concrete infill had nominal unconfined cylinder strengths ranging from 32 to 50 MPa. A semi-analytical finite strip method that included the beneficial effect of concrete was augmented to incorporate the true stress-strain behaviour and residual compressive and tensile stresses produced by welding. The model was then calibrated with the experiments conducted. The slenderness limits derived from the analysis were compared with Australian and British standards to illustrate the advantages obtained from a rational local buckling analysis. The effect of geometric imperfections was not included in the analysis, although the experimental results suggested they influenced the behaviour. A post-local buckling model based on the effective width principle was then established to determine the strength of CFST box columns. Uy (1998) suggested use of the AS4100 method, based on the effective width principle, for determining the post-local buckling behaviour of a column under pure compression for use in an ultimate strength analysis. However, Uy (1998) suggested that the results obtained provided further confirmation of the conservatism for design using the AS 4100 model.

Uy (2000) performed an extensive set of experiments to examine the effect of plate slenderness limits on the behaviour of short columns under the combined actions of axial compression and bending moment. The majority of test specimens had an *L/D* ratio of 3 and a *D/t* ratio between 40 and 100. Concrete strengths used in the test specimens achieved mean compressive strengths between 32 to 50 MPa. The results suggested that local buckling is significant in thin-walled CFST columns. Furthermore, the results indicated that the use of mean concrete compressive strength for maximum compressive concrete strength was found to be valid when the plate slenderness was compact. Uy (2000) noted that this may be due to the good quality of concrete caused by the retention moisture. It was further suggested that since the columns were not subjected to long term loads the value of the full compressive strength may have been appropriate. However, Uy (2000) then suggested that since researchers have found both final creep and shrinkage to be lower in CFST columns, the use of the mean compressive strength may have been suitable. A numerical model developed elsewhere is augmented and calibrated with these results. A simple model for the determination of the strength-interaction diagram was also verified against both the test results and the numerical model. The model, based on the rigid plastic method of analysis, is present in international design codes but does not account for the effects of local buckling. These effects were found to be significant with large plate slenderness values, particularly for members subjected to large axial load, and should be included in the modified rigid plastic analysis. Therefore, some suggested modifications were proposed to allow for the inclusion of thin-walled columns in design.

Uy (2001) investigated the local and post-local buckling behaviour of welded box sections. An extensive set of experiments was conducted on both hollow and CFST box sections. The *B/t* ratios ranged from 120 to 180. The tests for the hollow sections are significantly greater than the yield slenderness limits designated in international design codes. The nominal concrete strength was 20 MPa. However, the material properties of the concrete were unimportant as the concrete was merely used a restraint against local buckling and the concrete itself was not loaded. The residual stresses were measured and were very influential in the local buckling of the steel sections in the elastic range. Initial imperfections were not measured in the specimens as it was felt that they would be highly variable. The local buckling stresses were determined from the load-strain curves by noting changes in the stiffness. The results illustrated the potential increase in both the initial local buckling load and ultimate load, which can be derived from the inclusion of the concrete infill. A numerical model based on the finite strip method was used to determine the initial local buckling stress incorporating residual stresses. The model was shown to compare well with the experimental results. Furthermore, an effective width model developed elsewhere and presented in existing international design codes was augmented and calibrated with the test results. This method was found to be useful in determining the axial compressive strength of welded box sections (Uy 2001).

Bridge and O'Shea (1995) conducted a preliminary set of experiments to determine the effects of local buckling on the axial load behaviour of square thin-walled steel tubes with concrete infill. The steel was not bonded to the concrete infill which only provided restraint to inward local buckling. The *B/t* ratios ranged from 37.4 to 130.7 and the *L/B* ratio was 3.45. The nominal concrete strength was 20 MPa. The results indicated that the local buckling strength and ultimate strength of square steel tubes can be improved by providing unbonded concrete infill, especially for steel tubes with slender plate elements. Furthermore, the axial stress-strain behaviour of the steel plates restrained by concrete infill was established for square tubes. Bridge and O'Shea (1995) suggested that by using the strain compatibility in conjunction with the axial stress-strain behaviour of the concrete, the axial load behaviour of tubes with concrete bonded to the steel and loaded compositely could therefore be determined. The experimental strengths were compared with the design standard AS4100 (1990). The experimental results suggested that the approach used by AS4100 is conservative for tubes with and without restraint. Additionally, it is suggested that the enhancement in strength due to confinement of the concrete core in square CFST columns can be taken into account by using an appropriate elastic buckling coefficient in AS4100.

Bridge and O'Shea (1998) conducted a comprehensive set of tests to examine the behaviour of short thin-walled steel tubes subjected to axial load. Two series of test were preformed. In series 1, the effect of changing the buckling mode by forcing the formation of outward buckling was examined for a wide range of *B/t* ratios. In series 2, the influence of specimen length on the formation of local buckling was examined by using thin-walled steel tubes with and without internal restraint. The *B/t* ratios ranged from 37.3 to 130.7 and the *L/B* ratios ranged from 0.77 to 2.91. The nominal concrete strength was 20 MPa. The material properties, residual stresses and geometric imperfections were measured. The results indicated that the buckling mode was essentially the same for the range of *L/B* ratios and therefore the specimen length had little effect on the formation of local buckling. Furthermore, the results showed that the local buckling strength and ultimate strength of square steel tubes can be improved by providing internal lateral restraint, especially for thin-walled steel tubes. Importantly, Bridge and O'Shea (1998) stated that a buckling coefficient of 9.99 is appropriate when using the design provisions in AS 4100 for the design of square thin-walled CFST columns. The tests strengths were compared with strength models in design standards and recommendations for improved design were also made.

2.4 Design Codes

Currently there is no comprehensive design standard that can be utilised for the design of thin-walled CFST columns. Compact limits for most international design codes limit the slenderness values to less than 40 (Uy 2000). However, some guidance for the design of CFST columns is provided by the European Committee for Standardisation, the American Concrete Institute, and the Chinese Code. The design codes are based on several different theories, which can produce different results, and the assistance provided in terms of application varies significantly. A number of design standards take local buckling into account through the use of an effective diameter or an effective area method (O'Shea & Bridge 2000).

2.4.1 European Committee for Standardisation (Eurocode 4)

Eurocode 4 is the most recently completed international standard in composite construction (Giakoumelis and Lam 2004). The code is based on the rigid plastic method of analysis which assumes fully crushed concrete and fully yielded steel. The approach allows the full mean compressive strength of the cylinder to be utilised. The code uses a column curve, similar to most modern steel design codes, to determine the influence of slenderness in CFST columns. Local buckling is ignored by limiting the plate slenderness to within compact plate limits (Shanmugam & Lakshmi 2002). The enhancement of the concrete due to confinement is included for some specific cases and it is the only code that separately treats the effects of long-term loading. Furthermore, Eurocode 4 uses limit state concepts to achieve the aims of serviceability and safety by applying partial safety factors to load and material properties. The code is appropriate for the design of thick-walled steel tubes filled with normal strength concrete (O'Shea & Bridge 2000).

2.4.2 American Concrete Institute (ACI 318)

The code ACI318 uses the traditional reinforced concrete approach, with a minimum load eccentricity used to determine the column strength under nominal axial load. The American Standard excludes the influence of concrete confinement by the steel tube and accounts for column slenderness by a minimum load eccentricity. The limiting thickness of the tube to prevent local buckling is based on achieving yield stress in a hollow steel tube subjected to monotonic axial loading, which is not a necessary requirement for a CFST column (Giakoumelis and Lam 2004). The code ACI 318 is appropriate for the design of thick-walled steel tubes filled with normal strength concrete. The code ACI318 is different in concept to the Eurocode 4 (O'Shea & Bridge 2000). However, the Australian Standard (AS3600) uses a similar approach to the code ACI318.

2.4.3 Chinese Code (DL/T5085-1999)

The code DL/T5085-1999 is based on the unified theory that considers the CFST member as a composite member, as apposed to the separate components. The properties of CFST columns depend on the properties of the steel and concrete, and their dimensions. The composite indices and geometric properties are then used directly to obtain the ultimate strength. The Chinese code differs from both the Eurocode 4 and the code ACI318. The code also includes for shear and torsion, in addition to bending and axial load. Design by the code DL/T5085-1999 and Eurocode 4 give similar results for a column subjected to a high axial load and a small moment (Zhong & Goode 2001).

2.5 Summary

Considerable progress over the last 40 years has been made in the investigation of CFST columns. Fundamental knowledge on composite construction systems, such as ultimate strength, has already been obtained by the research conducted thus far (Shanmugam & Lakshmi 2001). However, intensive research is required, particularly for circular CFST members, on the interaction between the steel tube and concrete core, the effect of concrete restraining local buckling of steel plate elements, and the effect of concrete confinement.

While much of the current available research draws similar conclusions on the behaviour of CFST columns, there are a number of conflicting views being documented. The evidence of this is additional proof that further research is required into the behaviour of CFST columns. Currently there is no comprehensive design standard that can be used for the design of thin-walled circular CFST columns and further investigation is required before a new design standard can be introduced.

Methods of analysis, similar to some of those mentioned in this chapter, will be utilised in this dissertation to investigate the local and post-local buckling of steel tubes in thin-walled circular CFST columns.

Chapter 3 FINITE ELEMENT ANALYSIS

3.1 Introduction

This chapter will outline the technical details and development of the finite models used in this investigation. Three dimensional finite element models, created using the finite element code STRAND7 (2002), were used in this investigation to examine the critical local and post-local buckling of steel tubes in thin-walled circular CFST columns. A comprehensive set of design models was created for this investigation based on a standard model.

Geometric and material nonlinear analysis of the steel tubes with initial imperfections was undertaken. The initial imperfections of steel tubes consisted of geometrical imperfections and residual stresses. A lateral pressure was applied to the surface of the tube to induce the initial out-of-plane deflections in the finite element model. The material stress-strain curve of steel tubes with residual stresses was modelled by using the Ramberg-Osgood formula.

To treat the material plasticity of steel tubes, the von Mises yield criterion was adopted and integrated through the thickness of the plate. The mesh used in the analysis was found to provide an acceptable degree of accuracy based on a sensitivity analysis. The finite element analysis method was used to determine the minimum elastic local buckling coefficient of clamped steel tubes in CFST columns. Furthermore, two possible failure modes for the steel tube were identified; outward local buckling and yield failure.

3.2 Finite Element Model

3.2.1 Specifications of Standard Model

The finite element code STRAND7 (2002) was used to develop the models required to examine the critical local and post-local buckling of steel tubes in thin-walled circular CFST columns.

An eight-node quadratic plate element, based on the Mindlin plate theory, was employed in all analyses to describe the buckling displacements and stress distributions of the tubes. The buckling displacements were required to examine the local buckling behaviour of the steel tubes and the stress distributions were used to investigate the development of post-local buckling.

An approximate 24x24 mm mesh was used in all analyses and was found to provide an acceptable degree of accuracy based on a sensitivity analysis. The material properties of the models included a, Young's modulus of 200 GPa, Poisson's ratio was 0.3 and yield strength of 300 MPa. The three dimensional nonlinear finite element models had a diameter of 500 mm and the diameter-to-thickness ratios ranged from 50 to 200. One edge of the circular tube was assumed to be clamped owing to the inward buckling restraint provided by the concrete core. The other edge of the circular tube was assumed to be fixed. All nodes within the tube had 6 degrees of freedom.

3.2.2 Standard Model Procedure

The procedure outlined below was followed in STRAND7 (2002) to create a standard model.

Create Nodes:

1. Select: Tools / Points and Lines / Ellipse (i.e.1/4 circle)
2. Unclick the Create Beams Box, Steps = 8
3. P2 (Y direction) = 250, P3 (X direction) = 250 (i.e. diameter = 500)
4. Select: Edit / Select / All / Tools / Copy / by Increment
5. Increments (Z) = 25, Repeat = 1
6. Select nodes 1, 3, 5, 7, and 9 / Tools / Copy / by Increment
7. Increments (Z) = 12.5, Repeat = 1

Create Elements:

1. Select: Create / Element
2. Type = Quad 8, create elements by joining nodes
3. Select: Edit / Select / by Group / select plate / Tools / Copy / by Increment
4. Increments (Z) = 25, Repeat = variable (assigns length)
5. Select: Edit / Select / by Group / Tools / Mirror
6. YZ Plane (N2) select node 9, ZY Plane (N3) select node 1
7. Select: Edit / Select / by Group / Tools / Subdivide
8. All Elements (A) = 4, Targets (Plate) = Quad8

Assign Restraints and Loading Conditions:

1. Select all nodes around the bottom of the tube
2. Select: Attributes / Nodes / Restraint / Fix
3. Select all nodes around the top of the tube
4. Select: Attributes / Nodes / Restraint / Z sym
5. Select: Attributes / Plate / Edge Pressure (i.e. compressive load)
6. Value = 300 (i.e. yield strength of steel)
7. Select all edges around the top of the plate
8. Select: Global / Load and Freedom Cases / New Case / Load Case 2
9. Select: Edit / Select / by Group
10. Select: Attributes / Plate / Face Pressure / Normal (i.e. lateral pressure)
11. Value = variable

Assign Material Properties:

1. Select: Property / Plate
2. Modulus = 200,000
3. Poisson's Ratio = 0.3
4. Select: Geometry / Membrane Thickness = variable
5. Select: Tables / Stress vs Strain / Ramberg-Osgood

The steps in the procedure that required variable values were modified until a comprehensive set of design models was created.

Figure 3.1 illustrates a finite element model subjected to axial edge pressure. The models developed were checked for accuracy by comparison with existing results.

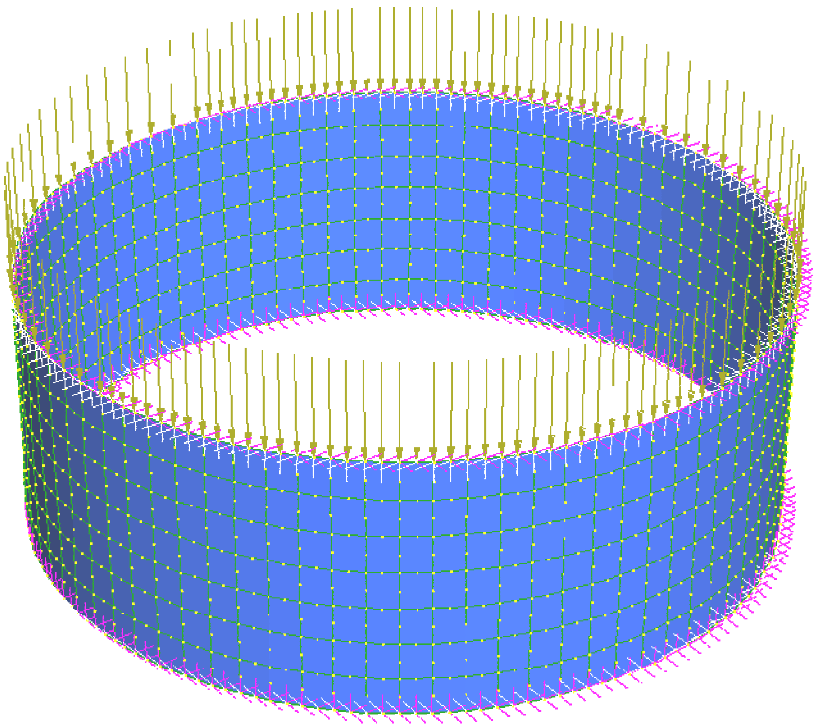

Figure 3.1: Finite element model subjected to axial edge pressure.

3.3 Sensitivity Analysis

A sensitivity analysis was conducted using STRAND7 (2002) to examine the degree of sensitivity of a model to changes in mesh size. The results from nonlinear static analysis of models with mesh sizes of 24x24 and 12x12 mm were compared. All dimensions and properties of the models examined, excluding the mesh size, were identical. The differences in mesh size are visibly evident by comparing

Figure 3.1 to Figure 3.2, the later having a mesh size of 12x12 mm.

A decrease in mesh size increases the total number of nodes and elements within a model and therefore increases the overall accuracy of the model. However, the size of the mesh is the governing factor behind the total time taken to complete analyses. It was necessary to conduct a sensitivity analysis to ensure that the results of the finite element investigation were of sufficient accuracy for use in engineering practise.

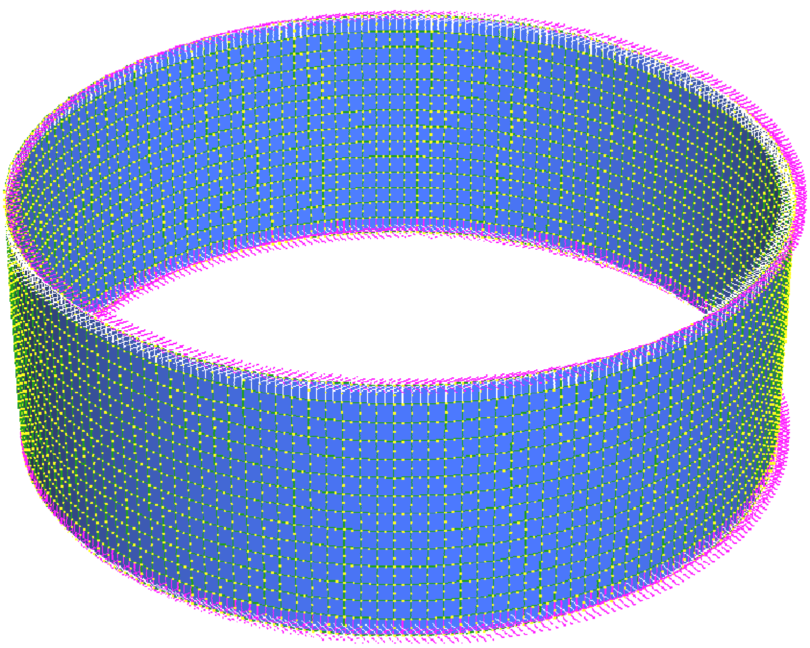

Figure 3.2: Finite element model with a mesh size of 12x12 mm.

3.4 Minimum Buckling Coefficient

The finite element analysis method, based on the bifurcation buckling theory, was used to determine the minimum elastic local buckling coefficient (k) of clamped steel tubes in CFST columns. The coefficient is dependent on the aspect ratio of the tube and on the boundary condition at its edge.

By using a Rayleigh-Ritz based method, Bradford et al. (2002) derived a closed form solution for the elastic buckling stress of circular hollow sections, see Equation (5). The value of the local buckling coefficient is absent from Equation (5) because it has already been assigned a value. Application of the formula is restricted by the slenderness ratio of the column.

$$\sigma_{0/} = \frac{2E}{\sqrt{3\left(1-\nu^{2}\right)}}\left(\frac{t}{d}\right) \tag{5}$$

The buckling coefficient, in the present study, was determined by conducting linear buckling analyses on tubes with various half-wavelengths (*h/D*). The linear buckling solver calculated the buckling coefficient and corresponding mode shapes for the steel tubes under the specified loading conditions. The minimum coefficients corresponding to mode shapes with a pattern of outward ring buckling, representative of the restraint offered by the concrete core, were recorded. When the aspect ratio *h/D* was equal to unity, the analysis produced a minimum buckling coefficient for the steel tubes.

The linear buckling analyses is based on the assumptions that there exists a bifurcation point where the primary and secondary loading paths intersect, and before this point is reached, all element stresses change proportionally with the elastic local buckling coefficient (STRAND7 2002).

3.5 Initial Imperfections

The initial imperfections of steel tubes consist of geometric imperfections and residual stresses, which are usually induced during processes of fabrication, transport, construction, and welding. Depending on their magnitude, initial imperfections have the potential to considerably reduce the strength and stiffness of steel tubes.

3.5.1 Geometric Imperfections

The effect of geometric imperfections on the local and post-local buckling of steel tubes in CFST columns was investigated using nonlinear finite element analysis. The STRAND7 (2002) nonlinear static solver uses an algorithm based on a modified Newton-Raphson method to predict the nonlinear behaviour of structures. A lateral pressure was applied to the surface of the tube to induce initial out-of-plane deflections in the finite element model.

Figure 3.3 illustrates a finite element model subjected to lateral face pressure. The magnitude of the lateral pressure was determined by undertaking nonlinear analysis of the model under trial loading. The model was subjected to only lateral pressure, no axial load. The lateral pressure that caused the maximum deflection at the centre of the tube equal to the specified deflection was recorded. A range of deflections from 0.05 to 0.25 mm were specified.

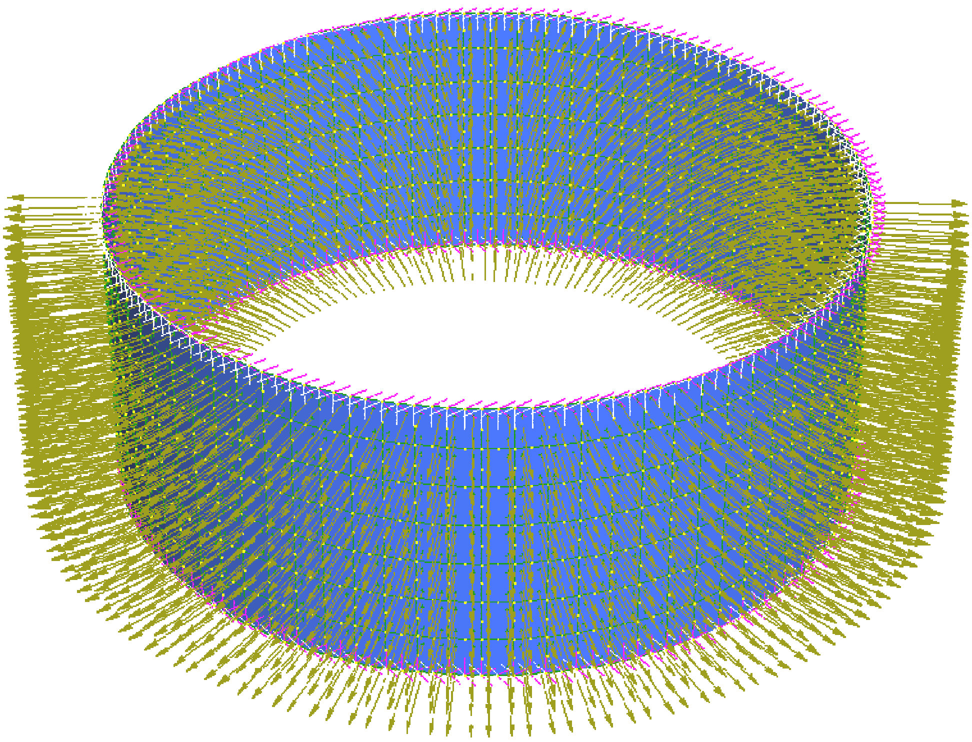

Figure 3.3: Finite element model subjected to lateral face pressure.

Previous research on CFST box columns by Liang and Uy (2000) specified the initial geometric imperfection as a linear function of the plate thickness. However, initial analysis of the standard model indicated that this method would not be suitable for use in the present study.

3.5.2 Residual Stresses

It was intended that the effect of residual stresses on the local and post-local buckling of steel tubes in CFST columns be investigated using the nonlinear finite element analysis method.

In previous research conducted by Liang and Uy (2000), the residual stresses in CFST box columns were treated by prestressing. A specified pre-load was combined with the applied edge stresses in the analysis. The results of the investigation indicated that residual stresses have a considerable effect on the stress–strain curve of a welded steel plate.

However, the incorporation of residual stresses into the modelling of circular steel tubes was deemed an issue too complex to address in the present study. This was largely due to the lack of existing finite element based research on circular CFST columns incorporating residual stresses.

3.6 Material Stress-Strain Curve

The material stress-strain curve of steel tubes in circular CFST columns is affected by residual stresses. The steel will yield when the sum of the applied stress and the compressive residual stress, caused by welding, is equal to the yield stress of the steel. After yielding, the stress-strain behaviour of the steel tube is no longer linear as would be the case for a steel tube under an equivalent applied load without residual stresses. The welded steel tube displays a rounded stress-strain form (Liang and Uy 2000), which can be seen in Figure 3.4.

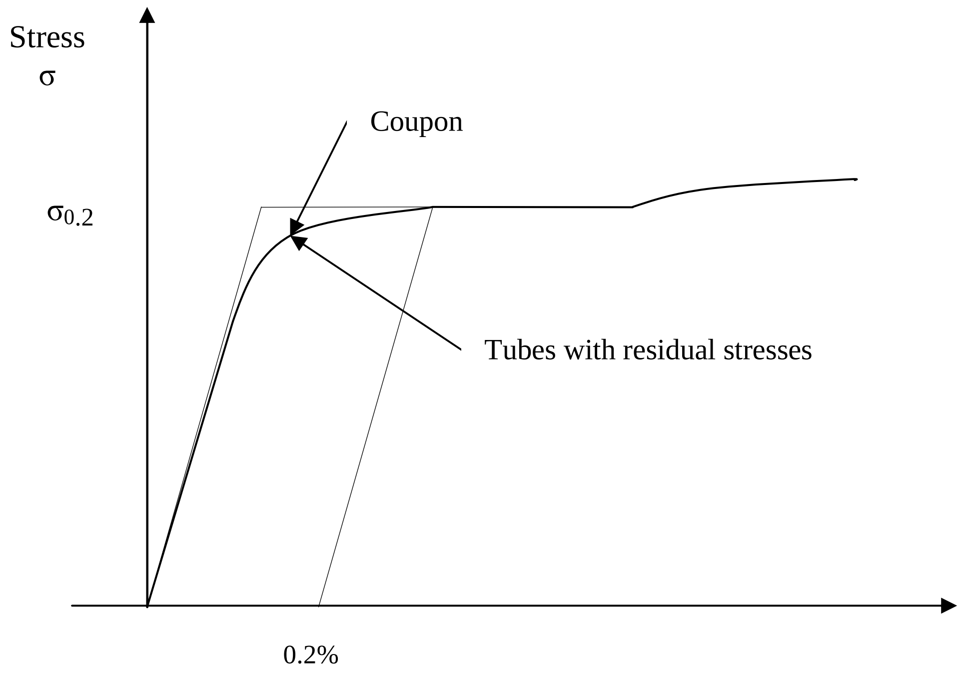

Figure 3.4: Material stress-strain curve of steel tubes with residual stresses.

The rounded stress-strain curve of steel tubes with residual stresses was modelled using the formula suggested by Ramberg and Osgood (1943), see Equation (6). The stress corresponding to $E_{0.7}$ ($\sigma_{0.7}$), used in the equation, was calculated to be 286.7 MPa.

$$\varepsilon = \frac{\sigma}{E} + \frac{3\sigma}{7E}\left(\frac{\sigma}{\sigma_{0.7}}\right)^{n} \qquad (6)$$

The knee factor (n) determines the sharpness of the knee of the stress-strain curve. For different materials the shape of the stress-strain curve can be defined by selecting a knee factor. It has been found that the knee factor n = 25 is suitable for structural mild steel with residual stresses (Moffin & Dwight 1984). A plot of the stress-strain curve based on the Ramberg-Osgood model can be seen in

Figure 3.5.

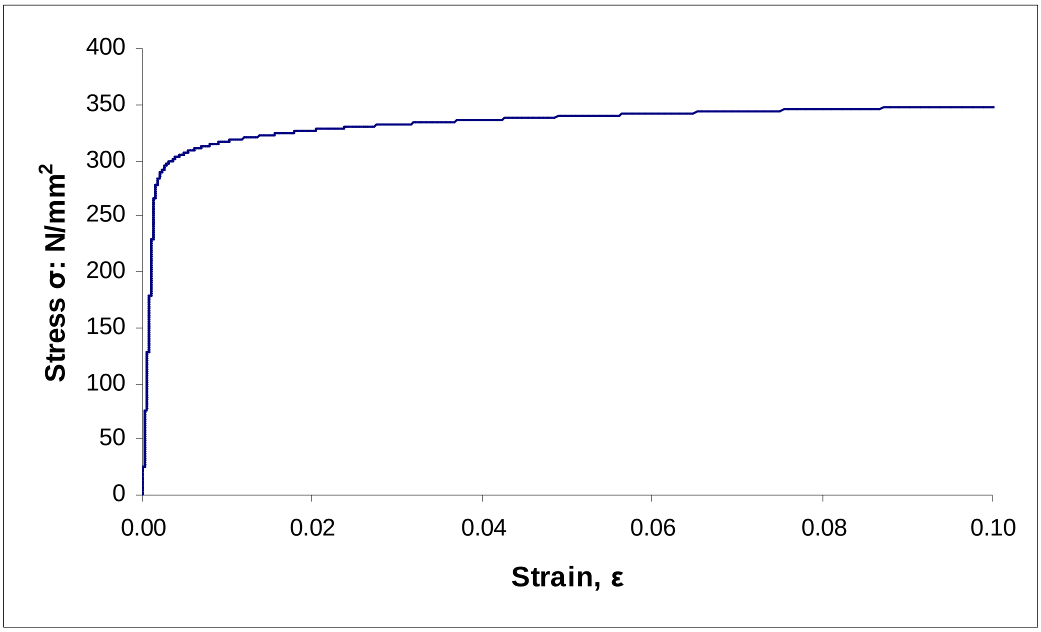

Figure 3.5: Stress-strain curve based on the Ramberg-Osgood model.

3.7 Von Mises Yield Criterion

To treat the material plasticity of steel tubes, the von Mises yield criterion was adopted and integrated through the thickness of the plate which was divided into 10 layers. As explained in STRAND7 (2002), the criterion states that yielding will commence when the second invariant of the deviatoric stress tensor $\left(J'_2\right)$ reaches the material parameter $K(k)$, see Equation (7). The material parameter $K(k)$ is a function of the hardening parameter k.

$$\left(J'_2\right)^{\frac{1}{2}} = K(k) \qquad (7)$$

Based on the yield criteria, the von Mises stress is defined by Equation (8).

$$\sigma_{vm} = \sqrt{\frac{1}{2}\left(\left(\sigma_{11}-\sigma_{22}\right)^2+\left(\sigma_{22}-\sigma_{33}\right)^2+\left(\sigma_{33}-\sigma_{11}\right)^2\right)} \qquad (8)$$

Where $\sigma_{11}, \sigma_{22}, \sigma_{33}$ are principal stresses such that $\sigma_{33} \langle \sigma_{22} \langle \sigma_{11}$.

3.8 Summary

The finite element code STRAND7 (2002) was used to develop three dimensional finite element models for investigating the critical local and post-local buckling of steel tubes in thin-walled circular CFST columns. The models developed were checked for accuracy by comparison with existing results.

Geometric and material nonlinear analysis of the steel tubes with initial imperfections was undertaken and the material stress-strain curve of steel tubes with residual stresses was modelled by using the Ramberg-Osgood formula. The von Mises yield criterion was adopted to treat the material plasticity of steel tubes. A sensitivity analysis was also conducted to ensure the accuracy of the finite element results. Furthermore, the finite element analysis method was used to determine the minimum elastic local buckling coefficient (k).

Chapter 4 DESIGN METHODOLOGY

4.1 Introduction

This chapter will discuss the methodology used to investigate the behaviour of CFST columns. A study of the nonlinear finite element analysis method was conducted in order to develop three dimensional finite element models for the geometric and material nonlinear analysis of steel tubes. The finite element analysis method was used to study the critical local buckling loads and post-local buckling reserve strengths of steel tubes in CFST columns.

Based on the results obtained, effective strength and ultimate strength formulas for the design of steel tubes in thin-walled circular CFST columns were developed using analytical methods. Furthermore, existing literature relating to the buckling behaviour steel tubes in circular CFST columns was researched with the aim of verifying the proposed formulas.

4.2 Finite Element Analysis

4.2.1 Critical Local Buckling Strength

The local buckling of steel tubes in CFST columns, subjected to axial compression, occurs when the applied load reaches the critical buckling load. It is important to understand the behaviour of local buckling in CFST columns because buckling of the steel tube can cause a reduction in the ultimate strength and stiffness of the member.

The finite element analysis method was used to study the critical local buckling strengths of steel tubes in thin-walled circular CFST columns subjected to uniform edge compression. The analysis was aimed at determining the influence of slenderness ratios and initial geometric imperfections on the local and post-local buckling strength of steel tubes in circular CFST columns.

No bifurcation point can be observed on the load-transverse deflection curves of steel tubes due to the presence of initial imperfections. This leads to difficulty in determining the initial local buckling loads of tubes with imperfections (Liang and Uy 2000). The method used for evaluating the critical local buckling loads of tubes with geometric imperfections was developed by Liang and Uy (2000) and is based on the load-transverse deflection relations associated with the theoretical analysis. The inflection point represents the maximum rate of the increment of transverse deflection with the load. The inflection point can be found by plotting the non-dimensional central transverse deflection verses the ratio of the deflection to the applied load w/σ_a. The minimum value of w/σ_a determined from the plot represents the inflection point where the local buckling occurs at the corresponding loading level (Liang and Uy 2000).

The transverse deflection verses w/σ_a curves were determined from the results of the finite element analysis. A contour plot of the displacement (DX axis) for a standard model with a displacement scale of 10% is shown in Figure 4.1. The results show a pattern of outward ring buckling commonly associated with steel tubes in CFST columns.

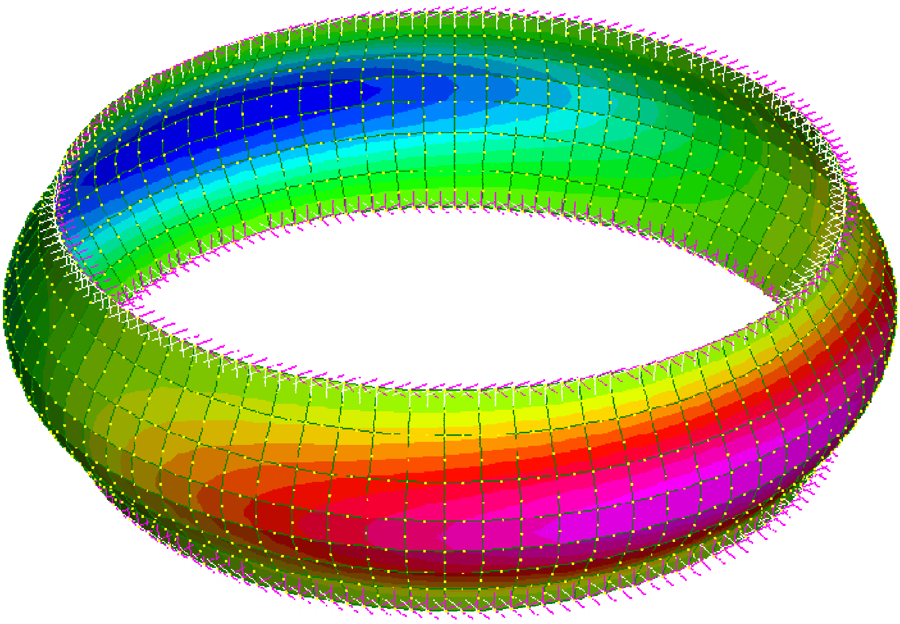

Figure 4.1: Contour plot of displacement (DX axis).

It was noted by Liang and Uy (2000) that for tubes with a low slenderness value, the inflection point might indicate yielding or plastic buckling because the sudden changes of the configuration of plastic deformations with loads that are only related to the yielding and plastic local buckling.

Theoretical studies indicate that the elastic local buckling stress for steel tubes in CFST columns is $\sqrt{3}$ times that of hollow steel tubes (Bridge et al. 1995). However, the available experimental data on the local buckling behaviour of steel tubes in CFST columns is inconclusive.

4.2.2 Post-Local Buckling Reserve Strength

After initial local bucking, thin-walled steel sections are still capable of carrying increased loads without failure. This behaviour of thin-walled steel sections is referred to as post-local buckling.

The development of the post-local buckling of steel plates is associated with the stress redistribution within the buckled plate under axial compression. The stress is redistributed towards the stiffer edges of the plates or plate assemblies and away from the more flexible regions of the plate (Bridge et al. 1995). However, this physical concept is more difficult to rationalise for axis-symmetrical circular tubes.

The stress distributions across the buckled section of tubes with geometric imperfections were obtained from the finite element analysis. A contour plot of the stress distribution (YY axis) for a standard model with a displacement scale of 10% is shown in Figure 4.2. The results suggest a stress redistributed towards the stiffer clamped edges of the tube.

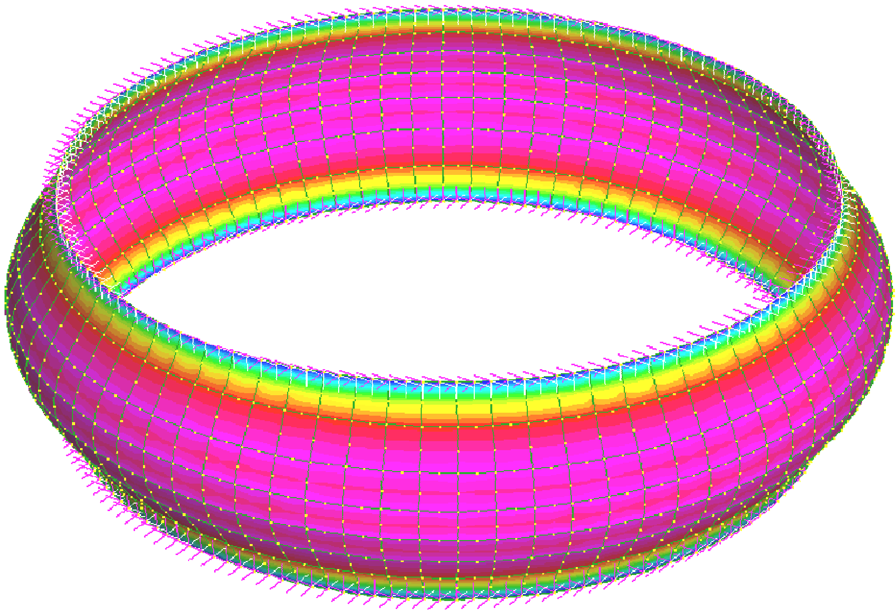

Figure 4.2: Contour plot of stress distribution (YY axis).

The finite element analysis method was used to study the post-local buckling reserve strengths of steel tubes in CFST columns. The post-local buckling reserve strength equates to the difference in the ultimate strength and the critical local buckling strength, see Equation (9).

$$\sigma_p = \sigma_u - \sigma_c \tag{9}$$

Available experiment research suggests that there is no post-local buckling reserve of strength in circular CFST columns, and therefore the critical buckling stress governs the strength of slender cross-sections. This suggestion was examined in the present study for circular steel tubes with geometric imperfections using the finite element analysis method.

4.3 Design Formulas

Based on the results obtained from the nonlinear finite element analysis, effective strength and ultimate strength formulas, incorporating local buckling effects, were proposed for the design of steel tubes in circular CFST columns. The design of steel tubes in circular CFST columns should account for restraint against local buckling provided by the concrete core. Both design formulas are a function of the slenderness ratio *D/t*. The formulas were verified by comparison with experimental results obtained by O'Shea and Bridge (1997).

4.3.1 Effective Strength Formula

Analytical methods were used to develop the effective strength formula based on the critical local buckling strengths of the steel tubes obtained from the transverse deflection verses w/σ_a curves. Using the software package MATLAB (2004), a polynomial was fitted to the critical buckling strength (σ_c) verses slenderness ratio curves *(D/t)*. The fitting process was generalised to determine the coefficients of a second-order polynomial that best fitted the 7 data points. The coefficients were calculated by the MATLAB (2004) function *polyfit*.

4.3.2 Ultimate Strength Formula

A method similar to the one used to calculate the effective strength formula was used to determine the ultimate strength formula for the design of steel tubes in circular CFST columns. Based on the ultimate strengths of the steel tubes obtained from the load-deflection curves, the MATLAB (2004) function *polyfit* was used to fit a second-order polynomial to the 7 data points.

4.4 Summary

There are two principal factors that make the determination of the critical local and post-local buckling behaviour complicated in this investigation; the non-linear material characteristics of both the steel and concrete, and the geometrical imperfections in the steel tube.

The finite element analysis method was used to study the critical local buckling loads and post-local buckling reserve strengths of steel tubes in circular CFST columns. Based on the results obtained, analytical methods were used to propose effective and ultimate strength formulas for the design of steel tubes in circular CFST columns. Furthermore, the formulas were verified by comparison with existing experimental results.

Chapter 5 RESULTS AND DISCUSSION

5.1 Introduction

This chapter will present and discuss the results that were obtained from the nonlinear finite element analysis conducted to investigate the local and post-local buckling of steel tubes in circular CFST columns. The results from the sensitivity analysis conducted and the linear buckling analysis that was performed to determine the local buckling coefficients are also presented.

The effects of geometric imperfections, slenderness ratios, and height-to-diameter ratios on the local and post-local buckling behaviour of steel tubes in CFST columns were considered in the analysis. This investigation was conducted with the use of load-deflection curves, load-edge shortening curves, and transverse deflection versus w/σ_a curves.

Furthermore, an effective strength formula and an ultimate strength formula are proposed for the design of steel tubes in circular CFST columns. The formulas are compared with the results of the finite element analysis and existing experimental data. Throughout the analysis process every attempt was made to ensure the accuracy of the results by comparison with existing studies.

5.2 Initial Analysis

5.2.1 Sensitivity Analysis

A sensitivity analysis was conducted to examine the degree of sensitivity of a model to changes in mesh size. The results from nonlinear static analysis of models with mesh sizes of 24x24 and 12x12 mm were compared. The steel tubes had a h/D ratio of 0.35 and the *D/t* ratios were 50 and 200. An axial pressure of 300 MPa and a geometric imperfection of 0.15 mm were applied to all models. All dimensions, properties, and loading conditions of the models, excluding the mesh size, were identical. Figure 5.1 presents a comparison of the load-deflection curves attained from the analysis results.

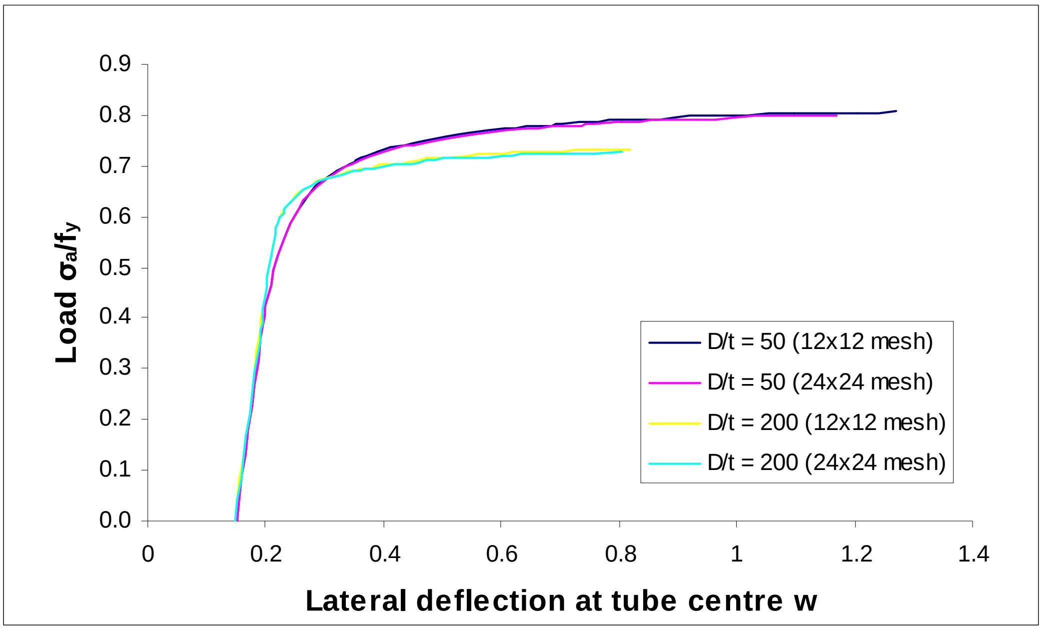

Figure 5.1: Load-deflection curves for sensitivity analysis.

The results in Figure 5.1 indicate that there is very minimal difference between the load-deflection behaviour of models with different mesh sizes. A comparison of the ultimate strengths of the steel tubes with different mesh sizes is presented in Table 5.1.

Table 5.1: Ultimate stress for sensitivity analysis.

D/t Ratio	12x12 σ_u (MPa)	24x24 σ_u (MPa)	σ_u/σ_u (%)
50	219.90	217.90	0.91
200	241.83	240.00	0.76

Table 5.1 shows that the ultimate stress of a steel tube is only slightly affected by the size of its mesh. The results in Table 5.1 indicate that a steel tube with a *D/t* ratio of 200 and a mesh size of 24x24 mm has an ultimate stress of 240 MPa. When the mesh size is decreased, the ultimate stress is only 0.76% higher.

Therefore, the 24x24 mm mesh was considered to provide an acceptable degree of accuracy for use in this investigation.

5.2.2 Minimum Buckling Coefficient

Linear buckling analysis was conducted to determine the minimum buckling coefficient (*k*) of steel tubes in CFST columns. The models had a *D/t* ratio of 100 and the *h/D* ratios ranged from 0.2 to 0.45. The steel tubes were subjected to an axial pressure of 300 MPa. All dimensions, properties, and loading conditions of the models, excluding the height, were identical. The coefficients recorded correspond to a mode shape of outward ring buckling. Figure 5.2 presents the buckling coefficients attained from the analysis.

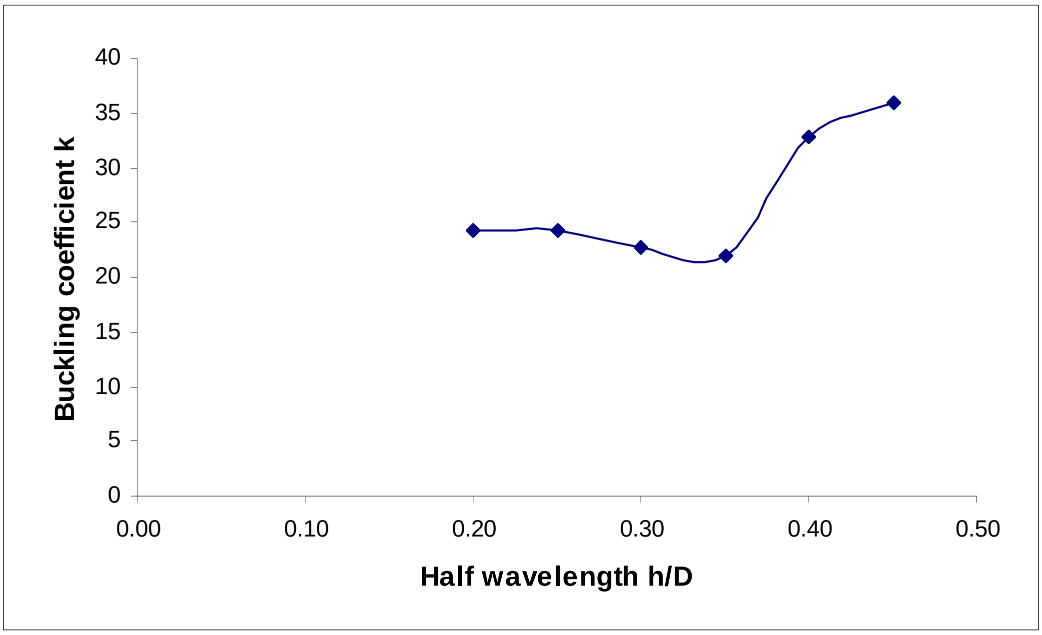

Figure 5.2: Buckling coefficient verses half wavelength.

Figure 5.2 indicates the analysis produced a minimum buckling coefficient of 21.85 for steel tubes in circular CFST columns when the aspect ratio *h/D* was equal to unity. Whereas, a study by Liang and Uy (1998) produced a minimum buckling coefficient of 9.81 for steel plates in CFST box columns. The minimum coefficient corresponded to a *h/D* ratio of 0.35. Based on the result obtained, the standard models used in further analysis have a diameter of 500 mm and a height of 175 mm.

5.3 Effects of Geometric Imperfections

The effect of geometric imperfections (δ) on the local and post-local buckling behaviour of steel tubes in CFST columns was investigated with the use of nonlinear static analysis. The results from the analysis were used to determine load-deflection curves and transverse deflection versus w/σ_a curves. The ultimate stresses and critical local buckling loads obtained from these curves were used to determine the post-local buckling reserve strengths for steel tubes in CFST columns. An axial pressure of 300 MPa and geometric imperfections ranging from 0.05 to 0.25mm were applied to the steel tubes. The models had a *h/D* ratio of 0.35 and a *D/t* ratio of 175. All dimensions, properties, and loading conditions of the models, excluding the geometric imperfection, were identical

5.3.1 Load-Deflection Curves

The load-deflection curves attained from the results of the nonlinear static analysis were used to investigate the effects of geometric imperfections on the local buckling behaviour of steel tubes in CFST columns. A comparison of the load-deflection curves for steel tubes with various geometric imperfections is presented in Figure 5.3.

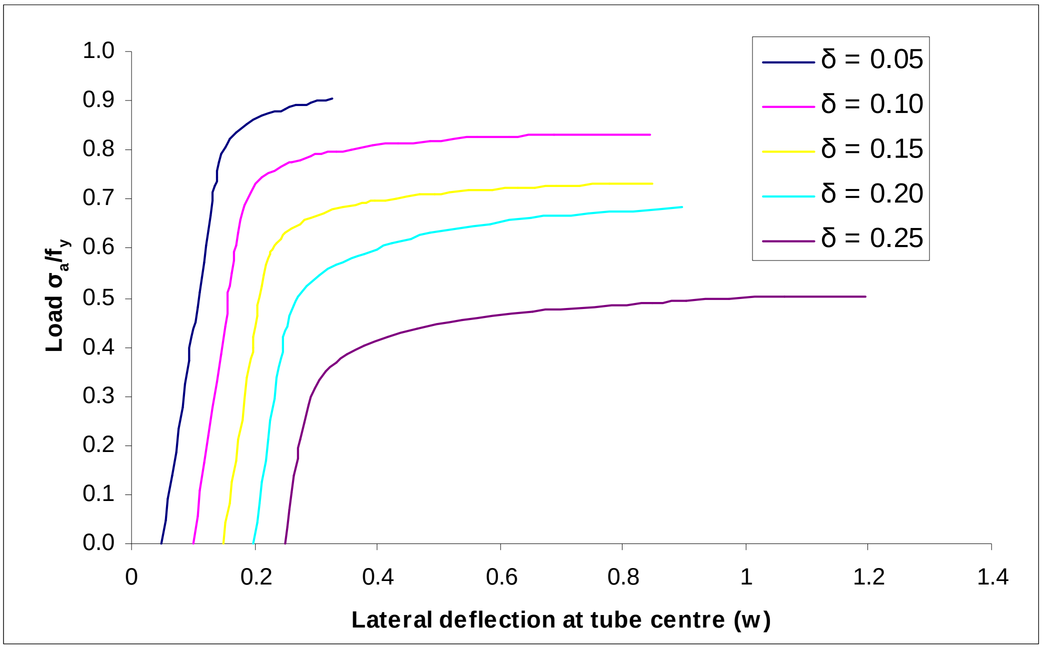

Figure 5.3: Load-deflection curves for tubes with different geometric imperfections.

It can be seen from Figure 5.3 that the tubes considered in the analysis could not attain their yield strength because of the geometrical imperfections applied. Figure 5.3 also shows that for steel tubes with larger geometric imperfections the critical local buckling strength is lower than that of steel tubes with smaller geometric imperfections. For a geometric imperfection of 0.05, Figure 5.3 shows that for a lateral deflection at tube centre *w* of 0.6 mm the corresponding non-dimensional load σ_a/f_y is 0.93. When the geometric imperfection increases from 0.05 to 0.25 mm, the load that can be applied to produce the same deflection is 20.2% lower. The ultimate stresses for steel tubes with various geometric imperfections are presented in Table 5.2.

Table 5.2: Ultimate stresses for tubes with different geometric imperfections.

Imperfection (δ)	σ_u (MPa)	σ_u/f_y
0.05	271.56	0.91
0.10	249.98	0.83
0.15	219.40	0.73
0.20	204.75	0.68
0.25	151.20	0.50

It can be seen in Table 5.2 that for steel tubes with larger geometrical imperfections the ultimate stress is lower than that of steel tubes with small geometric imperfections. For a tube with a geometric imperfection of 0.05 mm, Table 5.2 shows that the ultimate stress is 271.56 MPa. However, when the geometric imperfection increases to 0.25 mm, the ultimate stress is 79.6 % lower. Therefore, the results indicate that local buckling affects the ultimate strength of steel tubes in CFST columns. Based on the results obtained, the standard models used in further analysis have a geometric imperfection of 0.15 mm.

5.3.2 Critical Buckling Coefficient

The transverse deflection verses w/σ_a curves attained from the load-deflection curves were used to determine the effects of geometric imperfections on the critical local buckling loads of steel tubes in CFST columns. A comparison of the transverse deflection verses w/σ_a curves for steel tubes with various geometric imperfections is presented in Figure 5.4.

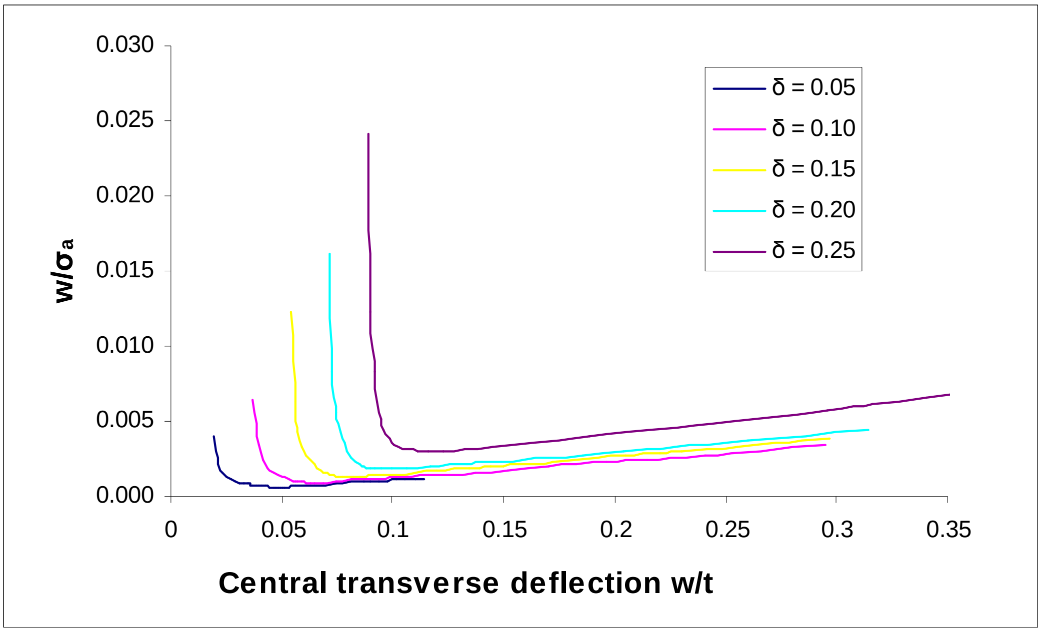

Figure 5.4: Transverse deflection verses w/σ_a curves for tubes with different geometric imperfections.

The results in Figure 5.4 indicate that for steel tubes with larger geometric imperfections the critical buckling loads are lower than that of steel tubes with higher geometric imperfections. Table 5.3 summarises the critical buckling loads of steel tubes with a range of geometric imperfections.

Table 5.3: Critical buckling loads for tubes with different geometric imperfections.

δ (mm)	w/t Ratio	w/σ_a Ratio	σ_c (MPa)
0.05	0.049	0.000616	227.9
0.10	0.065	0.000897	206.3
0.15	0.079	0.001285	176.4
0.20	0.095	0.001799	151.2
0.25	0.116	0.003014	110.3

The results presented in Table 5.3 show that for a steel tube with a geometric imperfection of 0.05, the corresponding critical buckling load is 227.9 MPa. When the geometric imperfection is increased to 0.25, the buckling load is 106.7 % lower. The results in Table 5.3 further demonstrate that the critical buckling loads of steel tubes are significantly affected by the magnitude of the geometric imperfection applied.

5.3.3 Post-Local Buckling Reserve of Strength

The ultimate stress and critical buckling loads obtained from the load-deflection and transverse deflection verses w/σ_a curves were used to determine the effects of geometric imperfections on the post-local buckling reserve of strengths of steel tubes in CFST columns. A comparison of the post-local buckling reserve of strengths for steel tubes with various geometric imperfections is presented in Table 5.4.

Table 5.4: Post-local buckling reserve strengths for tubes with different geometric imperfections.

Imperfection (δ)	σ_u (MPa)	σ_c (MPa)	σ_p (MPa)	σ_p/σ_u (%)
0.05	271.6	227.9	43.66	16.08
0.1	250.0	206.3	43.68	17.47
0.15	219.4	176.4	43.00	19.60
0.2	204.8	151.2	53.55	26.15
0.25	151.2	110.3	40.90	27.05

The results in Table 5.4 indicate that the post-local buckling strengths of steel tubes with larger geometric imperfections is higher than that of tubes with smaller imperfections. A post-local buckling reserve of strength of a tube with a geometric imperfection of 0.05 mm is 16% of its ultimate strength, whilst it is 27% of the ultimate strength of a plate with a geometric imperfection of 0.25 mm. Furthermore, the results suggest that steel tubes in CFST columns have high post-local buckling reserve strengths.

5.4 Effects of Slenderness ratio

The effect of slenderness ratios *(D/t)* on the local and post-local buckling behaviour of steel tubes in CFST columns was investigated with the use of nonlinear static analysis. The results of the analysis were used to determine load-deflection curves and transverse deflection versus w/σ_a curves. The post-local buckling reserve strengths for steel tubes in CFST columns were calculated based on the ultimate stresses and critical local buckling loads obtained from these curves. The models had a *h/D* ratio of 0.35 and the *D/t* ratios ranged from 50 to

200. An axial pressure of 300 MPa and a geometric imperfection of 0.15 mm were applied to the steel tubes. All dimensions, properties, and loading conditions of the models, excluding the *D/t* ratio, were identical.

5.4.1 Load-Deflection Curves

To investigate the effects of slenderness ratios on the local buckling behaviour of steel tubes in CFST columns, load-deflection curves were calculated from the results of the nonlinear static analysis. A comparison of the load-deflection curves for steel tubes with different slenderness ratios is presented in Figure 5.5.

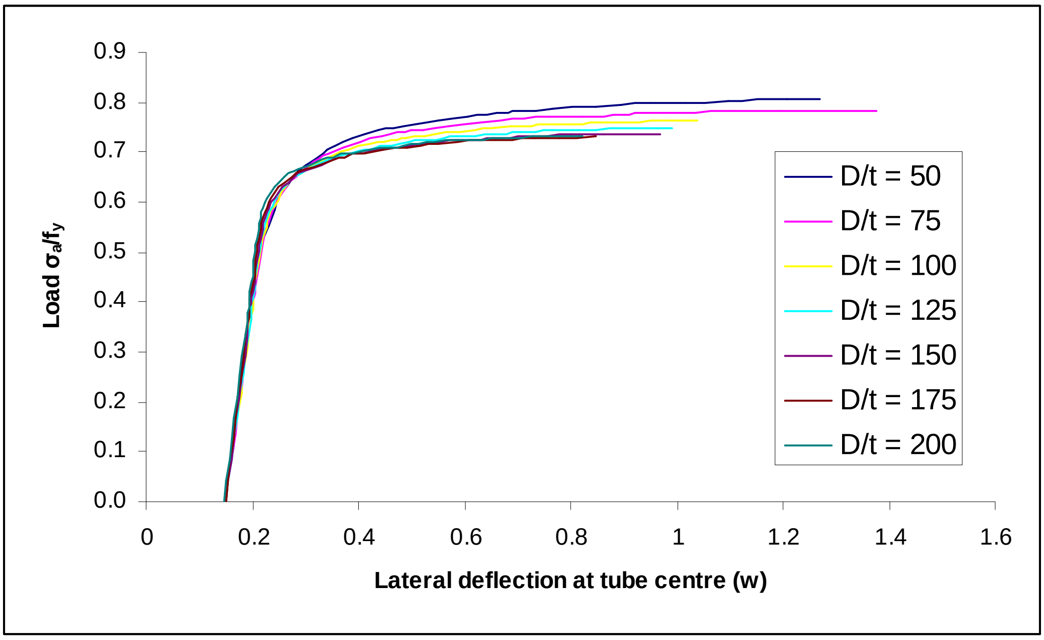

Figure 5.5: Load-deflection curves for tubes with different slenderness ratios.

Figure 5.5 shows that the tubes considered in the analysis could not attain their yield strength because of the geometrical imperfections applied. Figure 5.5 also shows that for steel tubes with larger slenderness ratios the critical local buckling strength is slightly lower than that of steel tubes with smaller slenderness ratios. For a slenderness ratio of 50, Figure 5.5 shows that for a lateral deflection at tube centre *w* of 0.6 mm the corresponding non-dimensional load σ_a/f_y is 0.77. When the slenderness ratio increases from 50 to 200 mm, the load that can be applied to produce the same deflection is 6.6% lower. The ultimate stress for steel tubes with various slenderness ratios is presented in Table 5.5.

Table 5.5: Ultimate stresses for tubes with different slenderness ratios.

D/t Ratio	σ_u (MPa)	σ_u/f_y
50	241.83	0.81
75	235.46	0.78
100	229.13	0.76
125	224.71	0.75
150	221.40	0.74
175	219.40	0.73
200	219.90	0.73

Table 5.5 indicates that the ultimate stress for smaller *D/t* ratios is higher than that of larger *D/t* ratios. The ultimate strength of a steel tube with a *D/t* ratio of 200 is 73.3% of its yield strength, whilst it is 81% of the ultimate strength for a steel tube with a *D/t* ratio of 50. Therefore, the results indicate that local buckling affects the ultimate strength of steel tubes in CFST columns.

5.4.2 Load-Edge Shortening Curves

To investigate the effect of slenderness ratios on the stiffness of steel tubes in CFST columns, load-edge shortening curves were calculated from the results of the nonlinear static analysis. A comparison of the load-edge shortening curves for steel tubes with different slenderness ratios is presented in Figure 5.6.

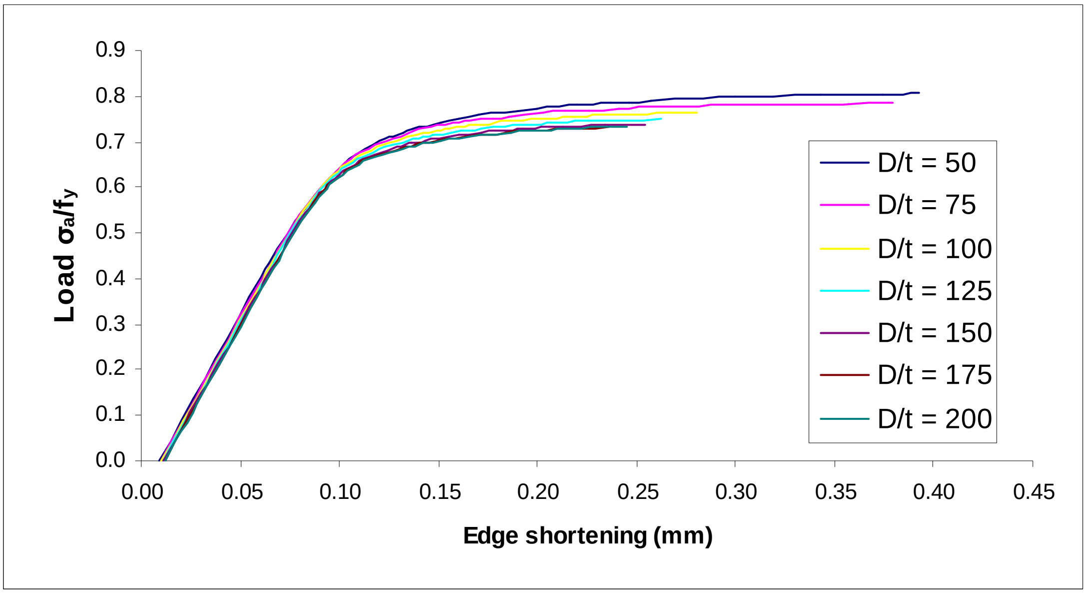

Figure 5.6: Load-edge shortening curves for tubes with different slenderness ratios.

The results from Figure 5.6 show that steel tubes with larger slenderness ratios have a lower stiffness when compared to tubes with smaller slenderness ratios. For a tube with a *D/t* ratio of 50, Figure 5.6 shows that for a non-dimensional load σ_a/f_y of 0.7 there is a corresponding edge shortening of 0.12 mm. When the *D/t* ratio increases from 50 to 200, the edge shortening that occurs is 18.5 % higher. The ultimate edge shortening of steel tubes with different geometric imperfections is presented in Table 5.6.

Table 5.6: Ultimate edge shortening for tubes with different slenderness effects.

D/t Ratio	Ultimate Edge Shortening (mm)
50	0.39
75	0.38
100	0.28
125	0.26
150	0.25
175	0.24
200	0.25

The results in Table 5.6 indicate that for steel tubes with larger slenderness ratios the ultimate edge shortening is lower than that of steel tubes with higher slenderness ratios. For a tube with a *D/t* ratio of 50, Table 5.6 shows that there is a corresponding ultimate edge shortening of 0.39 mm. When the *D/t* ratio increases from 50 to 200, the ultimate edge shortening that occurs is 56 % lower.

5.4.3 Critical Buckling Coefficient

The transverse deflection verses w/σ_a curves attained from the load-deflection curves were used to determine the effects of slenderness ratios on the critical local buckling loads of steel tubes in CFST columns. A comparison of the transverse deflection verses w/σ_a curves for steel tubes with various slenderness ratios is presented in Figure 5.7.

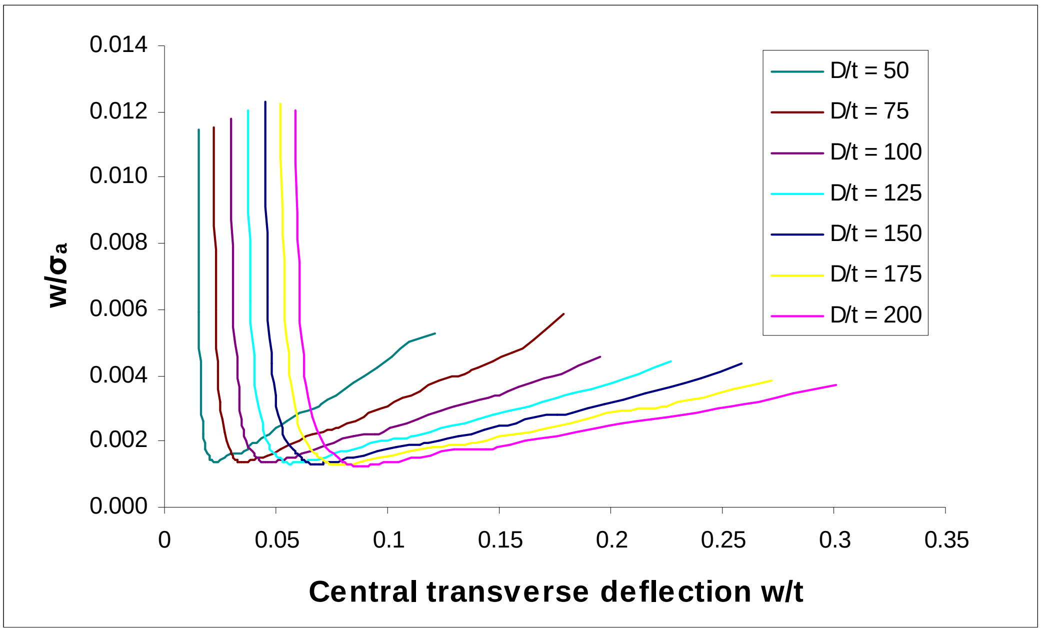

Figure 5.7: Transverse deflection verses w/σ_a curves for tubes with different slenderness ratios.

The results in Figure 5.7 indicate that for steel tubes with larger slenderness ratios the critical buckling loads are minimally lower than that of steel tubes with higher slenderness ratios. Table 5.7 summarises the critical buckling loads of steel tubes with a range of slenderness ratios.

Table 5.7: Critical buckling loads for tubes with different slenderness ratios.

D/t Ratio	w/t Ratio	w/σ_a Ratio	σ_c (MPa)
50	0.0241	0.001372	175.500
75	0.0360	0.001366	175.500
100	0.0481	0.001351	178.200
125	0.0580	0.001332	174.150
150	0.0694	0.001312	176.400
175	0.0793	0.001285	176.360
200	0.0897	0.001285	174.493

The results from Table 5.7 show that for a steel tube with a slenderness ratio of 50, the corresponding critical buckling load is 175.5 MPa. When the slenderness ratio is increased to 200, the buckling load is only 0.6% lower. The results in Table 5.7 indicate that the critical buckling loads are only minimally affected, if not at all, by the slenderness ratio of steel tubes in circular CFST columns.

5.4.4 Post-Local Buckling Reserve of Strength

The ultimate stress and critical buckling loads obtained from the load-deflection and transverse deflection verses w/σ_a curves were used to determine the effects of slenderness ratios on the post-local buckling reserve of strengths of steel tubes in CFST columns. A comparison of the post-local buckling reserve of strengths for steel tubes with various slenderness ratios is presented in Table 5.8.

Table 5.8: Post-local buckling reserve strengths for tubes with different slenderness ratios.

D/t Ratio	σ_u (MPa)	σ_c (MPa)	σ_p (MPa)	σ_p/σ_u (%)
50	241.8	175.5	66.33	37.79
75	235.5	175.5	59.96	34.17
100	229.1	178.2	50.93	28.58
125	224.7	174.2	50.51	29.00
150	221.4	176.4	45.00	25.51
175	219.4	179.6	39.80	22.16
200	219.9	174.5	45.40	26.02

The results in Table 5.8 indicate that the post-local buckling strengths of steel tubes with larger slenderness ratios are lower than that of tubes with smaller slenderness ratios. A post-local buckling reserve of strength of a tube with a slenderness ratio of 50 is 38% of its ultimate strength, whilst it is 22% of the ultimate strength of a plate with a geometric imperfection of 175. Furthermore, the results suggest that steel tubes in CFST columns posses high post-local buckling reserve strengths.

5.5 Effects of Height-to-Diameter Ratios

The effect of height-to diameter ratios on the local and post-local buckling behaviour of steel tubes in CFST columns was investigated with the use of nonlinear static analysis. The results from the analysis were used to determine load-deflection curves and transverse deflection versus w/σ_a curves. The ultimate stresses and critical local buckling loads obtained from these curves were used to determine the post-local buckling reserve strengths for steel tubes in CFST columns. The models had a *D/t* ratio of 175 and *h/D* ratios ranging from 0.25 to 0.45. An axial pressure of 300 MPa and a geometric imperfection of 0.15 mm were applied to the steel tubes. All dimensions, properties, and loading conditions of the models, excluding the *D/t* ratio, were identical.

5.5.1 Load-Deflection Curves

Load-deflection curves were calculated from the results of the nonlinear static analysis to investigate the effects of *h/d* ratios on the local buckling behaviour of steel tubes in CFST columns. A comparison of the load-deflection curves for steel tubes with different *h/d* ratios is presented in Figure 5.8.

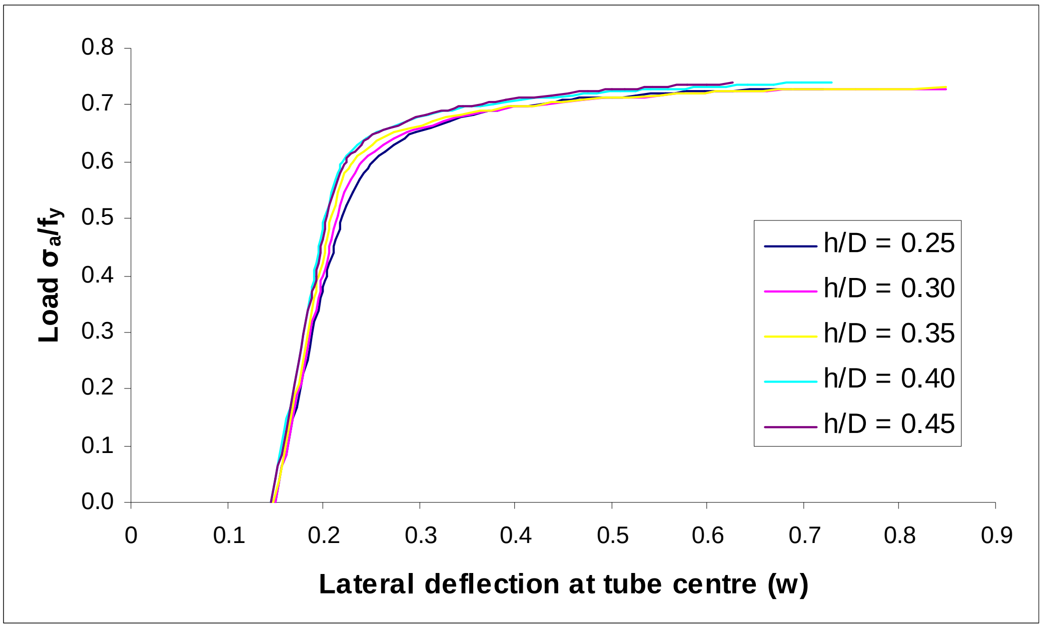

Figure 5.8: Load-deflection curves for tubes with different *h/D* ratios.

It can be seen from Figure 5.8 that the tubes considered in the analysis could not attain their yield strength because of the geometrical imperfections applied. The results in Figure 5.8 also show that for steel tubes with smaller *h/d* ratios the critical local buckling strength is slightly lower than that of steel tubes with larger *h/d* ratios. For a tube with a *h/D* ratio of 0.25, Figure 5.8 shows that for a lateral deflection at tube centre *w* of 0.3 mm the corresponding non-dimensional load σ_a/f_y is 0.66. When the *h/D* ratio increases from 0.25 to 0.45, the load that can be applied to produce the same deflection is 3.8 % higher. Table 5.9 presents the ultimate stresses for the steel tubes obtained from the results.

Table 5.9: Ultimate stress for different *h/D* ratios.

h/D Ratio	σ_c (MPa)	σ_c/f_y (%)
0.25	218.40	0.73
0.30	218.90	0.73
0.35	219.40	0.73
0.40	221.90	0.74
0.45	221.40	0.74

Figure 5.9 shows that the ultimate stress for smaller *h/D* ratios is minimally lower than that of larger *h/D* ratios. The ultimate strength of a steel tube with a *D/t* ratio of 200 is 74% of its yield strength, whilst it is 73% of the ultimate strength for a steel tube with a *D/t* ratio of 50. The results presented in Table 5.9 further suggest that the *h/D* ratio has little to no effect on the ultimate strength of steel tubes in CFST columns.

5.5.2 Critical Buckling Coefficient

The transverse deflection verses w/σ_a curves attained from the load-deflection curves were used to determine the effects of *h/D ratios* on the critical local buckling loads of steel tubes in CFST columns. A comparison of the transverse deflection verses w/σ_a curves for steel tubes with various is *h/D ratios* are presented in Figure 5.9.

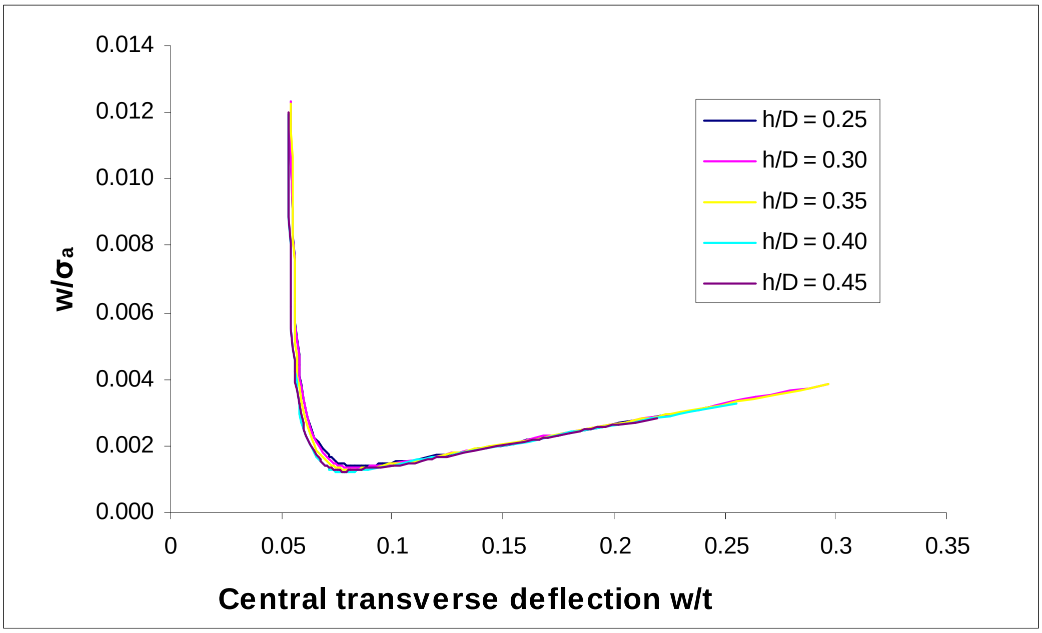

Figure 5.9: Transverse deflection verses w/σ_a curves for tubes with different *h/D* ratios.

The results in Figure 5.9 indicate that the critical buckling loads of steel tubes with larger *h/D* ratios are minimally lower than that of steel tubes with smaller slenderness ratios. Table 5.10 summarises the critical buckling loads of steel tubes with a range of *h/D* ratios.

Table 5.10: Critical buckling loads for tubes with different *h/D* ratios

h/D Ratio	w/t Ratio	w/σ_a Ratio	σ_c (MPa)
0.25	0.086	0.001399	176.4
0.30	0.083	0.001337	176.4
0.35	0.079	0.001285	176.4
0.40	0.077	0.001230	179.6
0.45	0.078	0.001245	179.6

The results from Table 5.10 show that for a steel tube with a *h/D* ratio of 0.25, the corresponding critical buckling load is 176.4 MPa. When the slenderness ratio is increased to 200, the buckling load is only 1.8% higher. The results in Table 5.10 contradict the suggested results in Figure 5.9. Nevertheless, both indicate that the *h/d* ratio has little to no affect on the critical buckling loads for the steel tubes analysed in this study.

5.5.3 Post-Local Buckling Reserve of Strength

The ultimate stress and critical buckling loads obtained from the load-deflection and transverse deflection verses w/σ_a curves were used to determine the effects of *h/d* ratios on the post-local buckling reserve of strengths of steel tubes in CFST columns. A comparison of the post-local buckling reserve of strengths for steel tubes with various slenderness ratios is presented in Table 5.11.

Table 5.11: Post-local buckling reserve strengths for tubes with different *h/D* ratios.

h/D Ratio	σ_u (MPa)	σ_c (MPa)	σ_p (MPa)	σ_p/σ_u (%)
0.25	218.4	176.4	42.00	23.81
0.3	218.9	176.4	42.50	24.09
0.35	219.4	176.4	43.00	24.38
0.4	221.9	179.6	42.30	23.55
0.45	221.4	179.6	41.80	23.27

The results in Table 5.11 indicate that the *h/D* ratio does not significant affect the post-local buckling strengths of steel tubes in CFST columns. However, the results do show that maximum post-local buckling strength and minimum buckling coefficient both correspond to a *h/D* ratio of 0.35. Furthermore, the results suggest that steel tubes in CFST columns have high post-local buckling reserve strengths.

5.6 Effective Strength Formula

5.6.1 Proposed Effective Strength Formula

The effective strength formula for the design of steel tubes in CFST columns is based on the results of the critical buckling strengths of steel tubes with geometric imperfections. Using the software package MATLAB (2004), a polynomial was fitted to the plot of the critical local buckling strengths of the steel tubes with geometric imperfections. The proposed effective strength formula can be seen in equation (10).

$$\frac{\sigma_c}{f_y} = -6.7\left(\frac{D}{t}\right)^2 e^{-7} + 1.5\left(\frac{D}{t}\right)e^{-4} + 0.58 \qquad (10)$$

The proposed formula is a function of the slenderness ratio. However in this study, the finite element results suggested that the slenderness ratio had a minimal to no effect on the critical local buckling strength of steel tubes in circular CFST columns.

5.6.2 Verification of Effective Strength Formula

The proposed formula was verified by comparing the results from the finite element analysis and experimental research. A comparison of the results obtained from the finite element analysis and the proposed effective strength formula can be seen in Figure 5.10.

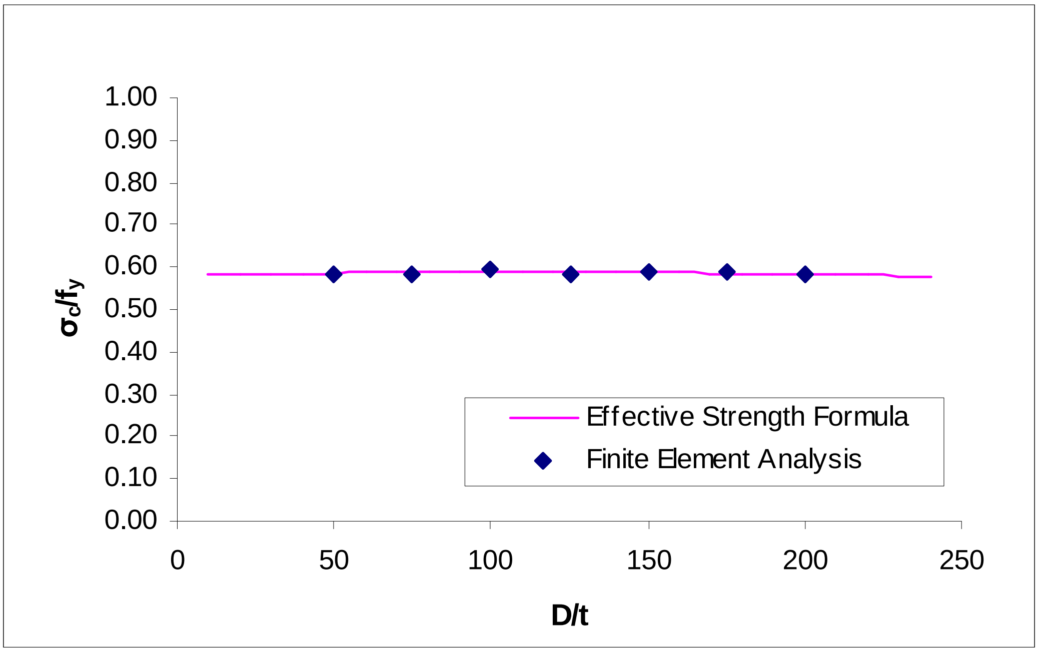

Figure 5.10: Comparison of critical buckling strengths obtained from FEA and proposed formula.

The results in Figure 5.10 indicate that the proposed formula fits accurately with the results of the finite element analysis. Furthermore, O'Shea and Bridge (1997) conducted four experimental tests on axially loaded steel tubes with concrete infill and the results can be seen in Table 5.12.

Table 5.12: Results of experimental research conducted by O'Shea and Bridge (1997).

D/t ratio	h/D Ratio	$\sigma_u/f_{y(0.2)}$
98	3.5	0.95
125	3.5	1.04
168	3.5	0.86
221	3.5	0.84

A comparison of the experimental results with the proposed effective strength formula is presented in Figure 5.11.

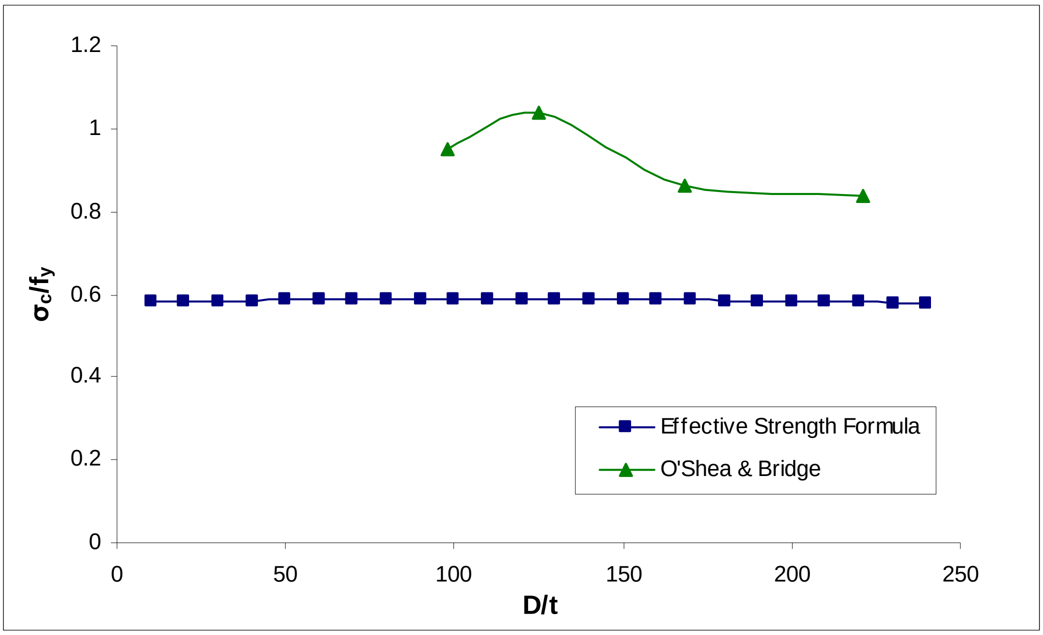

Figure 5.11: Comparison of effective strength formula and experimental data.

The results shown in Figure 5.11 suggest that the proposed effective strength formula, which is based on the critical local buckling loads, is conservative for the design of steel tubes in thin-walled circular CFST columns. However, the experimental results obtained by O'Shea and Bridge (1997) are based on the ultimate strength of steel tubes in CFST columns and therefore a conservative comparison result was expected.

5.7 Ultimate Strength Formula

5.7.1 Proposed Ultimate Strength Formula

The ultimate strength formula for the design of steel tubes in CFST columns is based on the results of the ultimate strengths of steel tubes with geometric imperfections. A polynomial was fitted to the plot of the ultimate strengths of the steel tubes with geometric imperfections using data-manipulation software. The proposed effective strength formula can be seen in equation (11). The proposed formula is a function of the slenderness ratio.

$$\frac{\sigma_u}{f_y} = -3.7\left(\frac{D}{t}\right)^2 e^{-6} + 1.4\left(\frac{D}{t}\right)e^{-3} + 0.87 \qquad (11)$$

5.7.2 Verification of Ultimate Strength Formula

The proposed formula was verified by comparison with results from the finite element analysis and experimental research. A comparison of the results obtained from the finite element analysis and the proposed effective strength formula can be seen in Figure 5.12.

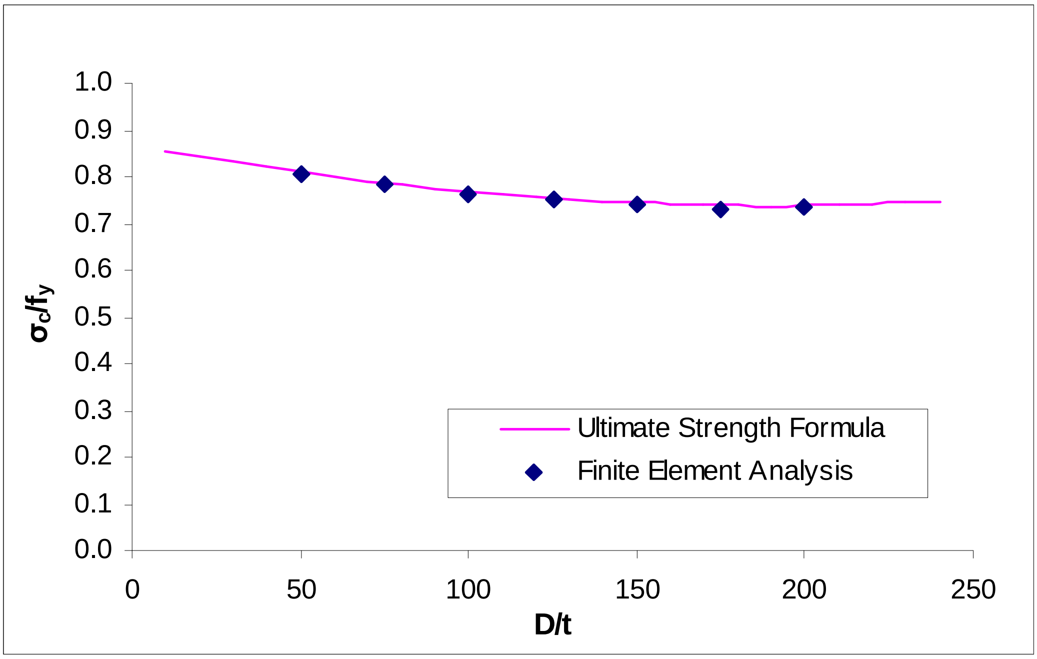

Figure 5.12: Comparison of ultimate strengths attained from FEA and proposed formula.

The results in Figure 5.12 indicate that the proposed formula fits precisely with the results of the finite element analysis. Once again, the experimental investigation conducted by O'Shea and Bridge (1997) is used to verify the ultimate strength formulas for the design of steel tubes in CFST columns. A comparison of the experiment results with the proposed ultimate strength formula is presented in Figure 5.13.

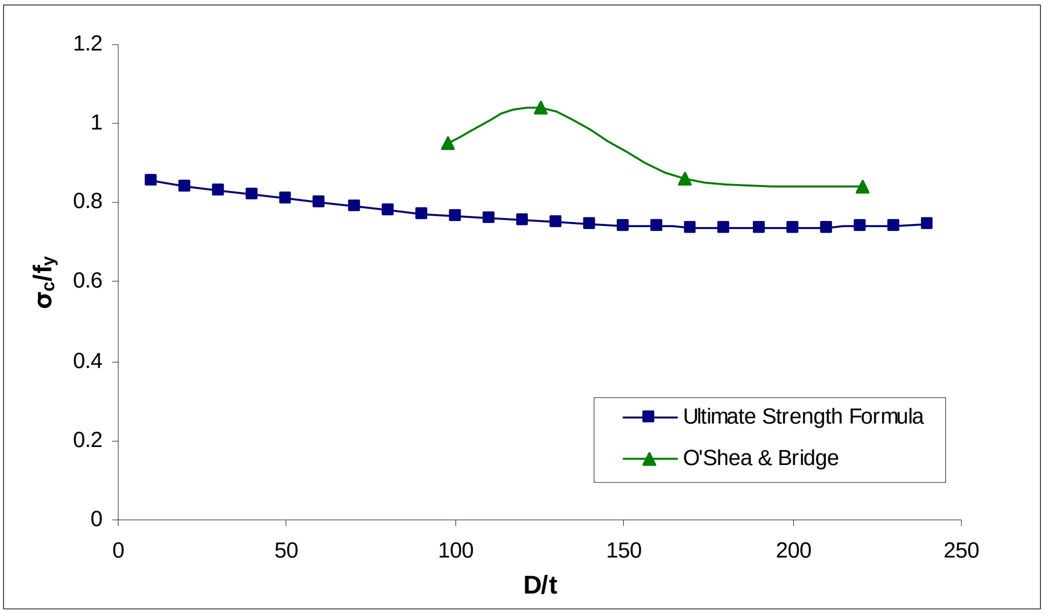

Figure 5.13: Comparison of ultimate strength formula and experimental data.

When comparing both sets of results, ignoring the experimental *h/d* ratio of 125, it can be seen from Figure 5.13 that for tubes with larger D/t ratios the ultimate stress is lower than that of steel tubes with small D/t ratios for both the formula and the experimental results. Looking at the experimental result for a thin-walled tube with a h/D ratio of 175, Figure 5.13 shows that the non-dimensional load σ_u/f_y is approximately 0.86 MPa. Furthermore, when using the formula the non-dimensional load σ_u/f_y 17.6% lower. This suggests that the ultimate strength formula, which is based on the ultimate strengths, is slightly conservative for the design of steel tubes in thin-walled circular CFST columns.

5.8 Summary

This chapter presented and discussed the results that were obtained from the nonlinear finite element analysis that investigated the local and post-local buckling of steel tubes in circular CFST columns. The effect of geometric imperfections, slenderness ratios, and height-to-diameter ratios were examined.

Based on the results, it was determined that the magnitude of the geometric imperfection, applied to the steel tube, has the most significant effect on the local and post-local buckling behaviour of steel tubes in CFST columns. In addition, the results suggest that both the slenderness ratio and height-to-diameter ratio have minimal effect on the local buckling behaviour of steel tubes. However, it was established that steel tubes in CFST columns have high post-local buckling reserve strengths.

Furthermore, an effective strength formula and an ultimate strength formula were proposed for the design of steel tubes in circular CFST columns. Both formulas compared accurately with the finite element analysis. When compared to available experimental data they appear to produce conservative results for both critical and ultimate load. However, while there is a suggestion that there was some correlation between the experimental results and the formulas, further data is required to assess the accuracy of both formulas before any suggestion of design application.

Chapter 6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

An investigation into the local and post-local buckling behaviour of steel tubes in circular CFST columns using the finite element method is presented. The effects of geometric imperfection, slenderness ratios, and height-to-diameter ratios on the critical buckling loads and ultimate strengths of steel tubes in CFST columns were considered. Effective and ultimate strength formulas for the design of steel tubes in CFST columns are also proposed.

6.2 Achievement of Aims and Objectives

This project aimed to investigate the local and post-local buckling behaviour of thin-walled circular CFST columns using the finite element method. The major objectives of this project accompanied with the outcomes of these aims are listed below:

1. Research existing information relating to local and post-local buckling of CFST columns.

The results of an extensive literature review are presented in Chapter 2. However, much of the research conducted is not aimed at local and post-local buckling of circular CFST columns columns.

2. Study the nonlinear finite element analysis method and develop three-dimensional finite element models for the nonlinear analysis of concrete-filled thin-walled steel tubular columns.

A study was completed of the finite element analysis method and Chapter 3 contains an outline the technical details and development of the finite element models used in this investigation.

3. Conduct geometric and material nonlinear finite element analysis on steel tubes under uniform/non-uniform edge compression to determine critical load and post-local buckling strengths. Initial geometric imperfections, residual stresses, material yielding and strain hardening will be considered in the analysis.

Geometric and material nonlinear finite element analysis on steel tubes under uniform edge compression was conducted and the critical load and post-local buckling strengths are discussed in Chapter 5. The analysis considers geometric imperfections, material yielding and strain hardening. Non-uniform edge compression and residual stresses were not considered.

4. Furthermore, investigate the effects of stress gradients and tube width-to-thickness ratios on the load-deflection curves and the post-local buckling strengths.

The effects of slenderness ratios were investigated and the load-deflection curves and post-local buckling strengths are discussed in chapter 5. The effects of stress gradients were not considered in the analysis.

5. Based on the results obtained; propose a set of design formulas for determining the critical local buckling loads and ultimate strengths of concrete-filled thin-walled steel tubular columns.

An effective strength formula for determining the critical buckling loads is proposed in Chapter 5. An ultimate strength formula for the design of thin-walled CFST columns was not proposed. However, an additional formula for determining the ultimate strength of steel tubes in CFST columns was developed.

6. Verify the proposed design formulas by comparison with existing experimental results and those developed by other researchers.

The proposed design formulas were verified by comparison with existing experimental results and are discussed in Chapters 5.62 and 5.72.

6.3 Conclusions

The results of the local and post-local buckling investigation indicated that the ultimate strength and stiffness of CFST columns is significantly affected by local buckling of the steel tube. It can also been seen from this investigation that steel tubes in CFST columns have high post-local buckling reserves of strength

Furthermore, the local and post-local buckling behaviour of steel tubes in CFST columns is significantly affected by the magnitude of the geometric imperfection implied. Initial imperfections affect the ultimate load, critical buckling load, and the post local buckling reserve of strength. Based on the results of this study, the slenderness ratio had a minimal effect on the critical buckling load of steel tubes in CFST columns. However, the ultimate load and post-local buckling reserve of strength for steel tubes in CFST columns was influenced by the slenderness ratio.

The effective strength formula proposed appears to be conservative when compared to the experimental results. However, the results are for ultimate strengths and it could be suggested that they are less than complete. The ultimate strength formula proposed attains similar result to the experiment data for slenderness ratios greater >150. However, while there is a suggestion that there was some correlation between the experimental results and the formulas, further data is required to assess the accuracy of both formulas before any suggestion of design application.

6.4 Recommendations

It is strongly recommend that further experimental and finite element based research be conducted on the critical buckling and ultimate strengths of circular CFST columns under various loading conditions. Currently, the information and data available is limited. A comprehensive understanding of the behaviour of circular CFST columns will not be achieved until this problem is rectified.

Other areas other research may also include:

- Concrete confinement.
- Interaction between the steel and concrete.
- Effect of eccentric loading conditions
- Fire resistance capacity
- The use of stiffeners in circular CFST columns

Appendix A

Project Specification

University of Southern Queensland

FACULTY OF ENGINEERING AND SURVEYING

ENG 4111/4112 Research Project
PROJECT SPECIFICATION

FOR: **Craig Louis Walker (Q11211742)**

TOPIC: Local and post-local buckling of concrete filled thin-walled steel tubular beam-columns.

SUPERVISOR: Dr Qing Quan (Stephen) Liang

ENROLMENT: ENG 4111 – S1, D, 2006
ENG 4112 – S2, D, 2006

PROJECT AIM: This project aims to investigate the local and post-local buckling behaviour of concrete-filled thin-walled steel tubular beam-columns using the finite element method.

SPONSOR: Faculty of Engineering and Surveying

PROGRAMME: **Issue A, 27th March 2006**

1. Research existing information relating to local and post-local buckling of concrete-filled thin-walled steel tubular beam-columns.

2. Study the nonlinear finite element analysis method and develop three-dimensional finite element models for the nonlinear analysis of concrete-filled thin-walled steel tubular beam-columns.

3. Conduct geometric and material nonlinear finite element analyses on steel tubes under uniform/non-uniform edge compression to determine critical load and post-local buckling strengths. Initial geometric imperfections, residual stresses, material yielding and strain hardening will be considered in the analysis.

4. Furthermore, investigate the effects of stress gradients and tube width-to-thickness ratios on the load-deflection curves and the post-local buckling strengths.

5. Based on the results obtained propose a set of design formulas for determining the ultimate strengths of concrete-filled thin-walled steel tubular beam-columns.

6. Verify the proposed design formulas by comparisons with existing experimental results and those developed by other researchers.

AGREED: (Student) (Supervisor)

(Dated) / / / (Dated) / / /

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